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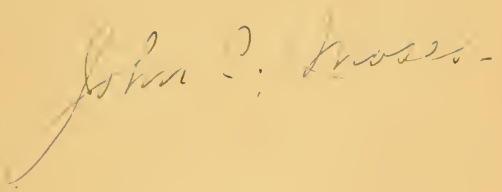

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXV.

JULY TO DECEMBER, 1891.

NEW YORK:
PUBLISHED BY THE SOCIETY.

1891.

Entered according to Act of Congress, by the American Society of Civil Engineers, in the Office of the Librarian of Congress at Washington.

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ERRATA.

By an exchange of tracings in preparing the matter for publication, the long scale to the right on Plates XCIX and C in the November number, page 537, was erroneously inserted.

The *correct* scale for each is that given under the title. The other is about 12 per cent. too small.

The numbers on Plates XCVI and XCVII should be reversed.

Volume XXV, p. 447, fifteenth line from top, "3 400 yards" should read "400 yards."

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(Vol. XXV.—July, 1891.)

CHIMNEY FOR THE NARRAGANSETT ELECTRIC LIGHTING COMPANY, PROVIDENCE, R. I.

By JOHN T. HENTHORN, M. Am. Soc. C. E.

WITH DISCUSSION.

The chimney for the Narragansett Electric Lighting Company of Providence, R. I., was designed by the writer's firm (Remington & Henthorn) and built under their supervision. It forms a part of a large central station, which was in process of construction at the same time under the supervision of the same designers.

The location of the center of the chimney is 101 feet west of the harbor line of the Providence River, and the foundation, which was begun in August, 1888, formed part of a general contract for other portions of the station. This foundation consists of piling and concrete, and to arrange for it, a space 44 feet square was first excavated 5 feet 6 inches below zero line or high water, and the sides protected by driving 3-inch spruce sheet-piling 16 feet long. Over this excavation the pile-driver, having a ram weighing 2 200 pounds, was rolled. Spruce piles, 50 feet long, and spaced 30 inches center to center, as shown on Plate I, were driven as far as possible without breaking. There are, however, many more piles shown on this sheet, but these were driven

to sustain other structures alongside, and are, therefore, not to be considered as chimney-piles. In the drawing, the chimney-piles, of which there are 231, are shaded slightly that they may be more readily recognized. These piles were cut off uniformly at 5 feet below the high-water line, the earth around their heads thus being 6 inches below their tops. The intervening space between the sheet-piling was filled in with concrete composed of one part of Norton's hydraulic cement, two parts sand, and three parts coarse gravel and broken stone. This mass was carried up to the 1 foot 3-inch level, and consequently formed a foundation 6 feet 9 inches thick, with the head of each pile projecting 6 inches therein. This was then covered with earth and allowed to season during the winter.

On May 31st, 1889, work was resumed by laying the first brick of the chimney. This was carried up in the form of a square of 36 feet to a height of 3 feet 2 inches, and from that level the base of the chimney proper which was 28 feet 6 inches square, was started.

The center of the chimney was fixed by building into the brick-work a cast-iron plate, upon which was a well-defined center mark. From this center mark all measurements and plumbing were established while the chimney was being built. As each 20 feet in height was built, the center of its axis was re-established, and if any deviation from the plumb was found it was corrected before the next 20-foot level was reached.

The base of the chimney, which, as before stated, is 28 feet 6 inches square, consists of three walls: an outer wall 28 inches thick; an intermediate wall, octagonal in form, 12 inches thick; and a core wall circular in section 16 inches thick. The outer and intermediate walls are joined by pilasters 12 inches thick.

In commencing the base of what might be termed the core wall, each course of brick was set back $2\frac{1}{2}$ inches from the previous course until the inside diameter, 14 feet, was reached, when the wall was carried plumb 16 inches in thickness up to the 78 feet 2-inch level, where it was reduced to 12 inches, and run up to 193 feet 2 inches, when it was again reduced in thickness to 8 inches, and thus carried to 249 feet 9 inches. This wall complete was laid in lime mortar, which had been slaked from three to six months before using. The outer wall, of rectangular cross-section, was carried up to the 38 feet 2-inch level, where at each of the four corners cut granite blocks were placed, to change from a square to an octagonal cross-section.

This granite work is made up of three pieces in the lower course of 2 feet 8 inches in thickness, and two courses with one stone each, also 2 feet 8 inches in thickness. On the angular face of these stones was cut a triangular section or projection to relieve the monotony of a plain surface, and when in place they fulfilled their mission to perfection. Level with the top of this granite work and in the center of the panel of the octagonal side is laid a terra cotta brick belt course, which thus forms the commencement of the 24-inch pilasters at each of the octagonal corners, and leaves an intervening panel 4 inches deep of varying width, as the chimney is reduced in size by the batter. These pilasters are carried up to the 223 feet 2-inch level, at which point the first projection for the head is made, and from that point carried still further, on the same batter, up to the crown of the arches, which are then turned between the panels.

The outside wall, together with the intermediate wall and their pilasters up to the 38 feet 2-inch level, were laid up with cement mortar, one part cement and two parts of sharp sand, excepting the outside course of the outer wall which was laid in soapstone finish mortar, of a dark red color. From this level (38 feet 2 inches) up to the commencement of the head, 223 feet 2 inches, the outer wall was laid up with mortar consisting of two parts lime, and one of cement. For laying up of the head from the 223 feet 2-inch level up to the point to receive the cast-iron cap, one part of lime and one of cement were used, and for backing up underneath the cap after it had been thoroughly screwed together, three of sand and one of Portland cement was used. Special brick was used for the outside course at each of the corners of the octagon, which were moulded to shape in order to avoid cutting and at the same time secure a better outside finish. The bond throughout the entire structure consisted of full headers for every six courses of brick-work. This method of construction was carried on up to the 66-feet 2-inch level when work was suspended November 29th, for the season of 1889, and the top of the structure covered with matched boarding and tarred paper for the winter.

The chimney was built entirely from the inside platform, the masons working overhanded, and thus no staging was necessary on the outside.

Up to the level of the granite work all the stock used was carried up a ladder placed on the outside. But at this point there was constructed inside the 14-foot chimney-flue, an elevator, fitted with safety-clutches

and capable of carrying 1 000 pounds, although not more than 450 pounds was allowed to be placed upon it at any one time; and thereafter everything used in the process of construction was sent up on the elevator, to hoist which a nineteen-strand steel cable was used. The temporary framework inside the flue consisted of four 6 x 8-inch timbers, laid across each other at right angles, in pairs, and built into the wall at intervals of every 5 feet. Through the opening at the center, the elevator passed. Over these timbers was laid a platform of 2-inch plank, upon which the masons performed their work. To these 6 x 8-inch horizontal timbers, at opposite corners, were bolted the vertical guides for the elevator and the upright framing by which it was hoisted; these were spliced out at the top each alternate staging.

The opening for smoke flues is 10 feet wide and 18 feet high, with a 28-inch arch of 5 feet radius. The lower part of the opening is on the 14 feet 2-inch level. Directly below and above the opening on the 13 feet 2-inch and 33 feet 2-inch levels, were placed in each of the four sides of the chimney and 8 inches from the outside surface of the wall —two 1¼-inch diameter tie-rods, with 1½-inch ends. These were all connected together by cast-iron corner-plates 12 x 14 inches square. Openings were left at each corner so that the nuts could be examined occasionally as the work dried out.

From and including the 53 feet 2-inch level, there were laid edgewise at each 20 feet in height and 8 inches from the outside surface of the chimney, wrought-iron bars of 4 inches x ½-inch, with their ends bolted together, forming an octagon corresponding to that of the chimney. At the 133 feet 2-inch and 153 feet 2-inch levels these braces were reduced in size to 3 inches x ½ inch, and were not again used until the 223 feet 2-inch level, or when the commencement of the head was reached, at which point bars 3 inches x ⅜ inch were bolted together in the wall. Their next application was in the head, where two braces made of 4 inches x ½-inch iron were used to assist in binding the heavy brick-work together during construction, which had considerable overhang (2 feet 9 inches on each side).

With reference to the outside wall, the outer and intermediate walls, with their connected pilasters, were built as one structure and terminated on the 83 feet 2-inch level, where by the batter of the outer wall it joined the intermediate and became one wall from that point. At this level two holes, 5 x 8 inches, were left in each of the eight sides

of the intermediate wall, so that the intervening space between the outer and inner walls might be ventilated, if by any possible chance gases should find access to this space. These ventilating holes are in communication with the space between the outer wall and the core, which is carried nearly to the top.

As before stated, work was suspended November 29th, 1889, and on April 22d, 1890, work was again resumed, but on account of bad weather it was thought well to hurry the construction along somewhat, so that June 28th four arc lights of 2 000 candle power, were placed upon the top of the elevator staging of the chimney for lighting at night, and a gang of masons relieved the day gang. This was continued until August 6th, 1890, when the night gang was dismissed, the chimney having then reached the height of 223 feet 2 inches. At this point the head of the structure was commenced, and gradually built up in cement mortar, as before stated, and measured when completed 24 feet across its sides; and on the 10th day of March, 1891, a flag was hoisted on its top by the eight-year-old son of a member of the firm of the designing engineers in honor of its completion, and in celebration of the visit of the National Electric Light Association, whose president, Mr. Marsden J. Perry, is also general manager and vice-president of the Narragansett Electric Lighting Company.

On October 4th, 1890, all brick-work was completed up to the 253 feet 9-inch level, and ready to receive the cast-iron cap, but on account of delays, the iron-work was not received to place in position until January 26th, 1891. On February 11th, the first of the twenty-five pieces of the cast-iron cap was hoisted to the top, and the work of bolting it together commenced. Copper bolts were used entirely for securing it together, these passed through flanges cast on the under side of the plate forming the cap. There were also on the outer surface upright flanges, one turning down over the other, and thus preventing water from penetrating the brick through the joints of the cap. This construction is plainly shown on Plate II. All of the joints of the cap where there was a possibility of leakage were carefully filled in with sal-ammoniac and iron borings, thoroughly rammed or calked in, and thus, as will be readily seen from the design of the joint (as shown in detail on Plate II), a perfectly tight and secure connection was obtained. The weight of the cap alone when in place was 22 000 pounds.

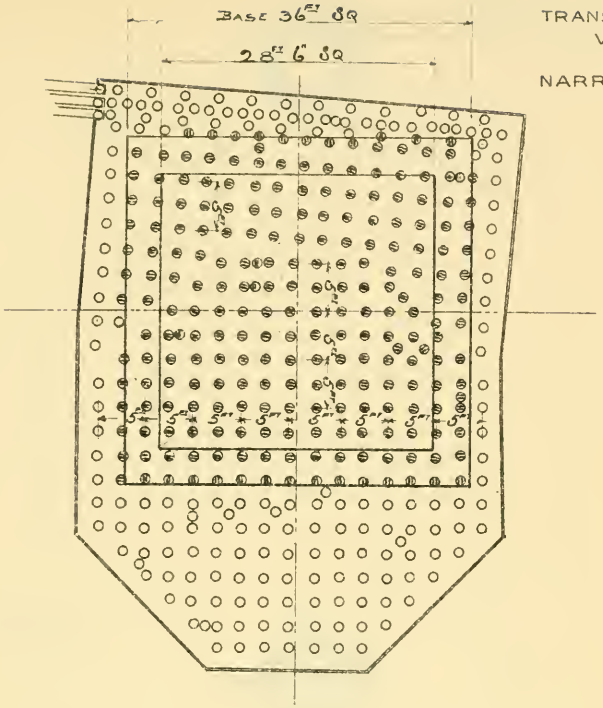
In order to facilitate ascending the chimney a ladder made of 1¼

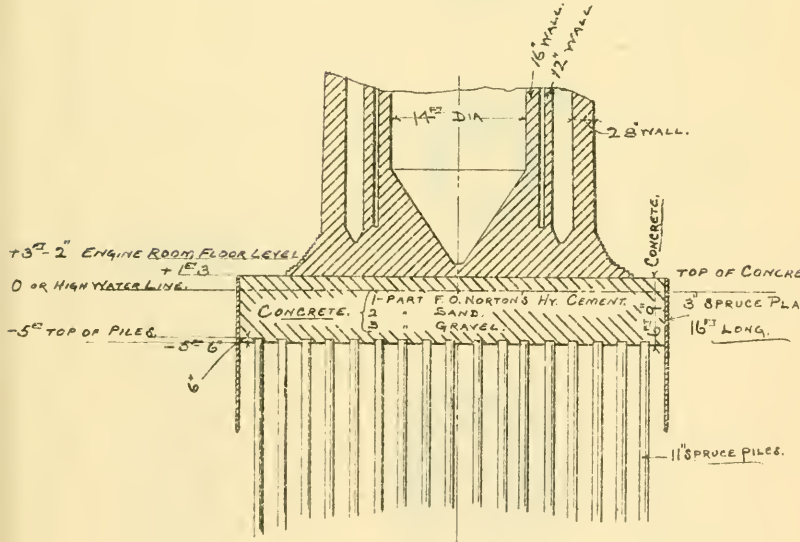
round iron was built solidly into the outer wall at a point in its circumference where there was the greatest space between the intermediate and core walls; these bars were spaced 16 inches center to center, and as before stated were built into the outer wall, while in the core wall their ends were merely inserted into a cavity, 2 inches wide and a course of brick high, so that any expansion of the inner core would not affect the solidity of the ladder. Ascending this ladder, provision is made for protection from the gases at the top by building across the corner a cast-iron vertical plate, with its edges turned into the brick-work 8 inches and its lower end terminating at a point 3 feet below the top of the core wall, thus forming a chamber; and from this, by means of a hatchway in the cap, also provided with ventilating openings in its side, access to the outside of the cap is readily had. Since the completion of the chimney this ladder has been used several times, not from necessity, but for the novelty of the deed on the part of workmen.

An attempt has been made to protect the structure from lightning by encircling the cast-iron cap with a copper ribbon 1 x $\frac{3}{16}$-inch thick, to which are connected by a riveted and soldered joint, eight brass upright sockets, one in the center of each panel of the cap. To these brass socket castings, are secured by soldered-joint, $1\frac{1}{4}$-inch seamless drawn copper tubing which extends upward above the top of the cap and conforms to its shape. After projecting 5 feet above the upper portion of the cap the tubes are each surmounted by a brass casting 28 inches long, tapering in cross-section and having at its extremity a platinum point $1\frac{3}{8}$ inches long. The encircling ribbon around the cap is connected with the ground ribbon by a brass casting thoroughly riveted and soldered thereto. As it runs down the chimney this ground ribbon is secured in position by brass clamps with bolts which were built into the brick-work as it progressed; this arrangement as a whole is shown in detail in Plate IV. The lower end of the ribbon, which is 1 x $\frac{3}{16}$-inch copper, rolled in one piece 285 feet long, terminates in a copper plate 30 inches wide by 60 inches long and $\frac{1}{16}$-inch thick, and is buried 4 feet below the natural level of the water in the soil of the premises. This plate is buried in a load of powdered coke, 18 inches being placed above and 18 inches in thickness below the plate, and the whole filled up with gravel.

After completion, the outside of the chimney was thoroughly oiled, in order to bring the brick-work to one uniform color.

PLATE I
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. No 483.
HENTHORN ON
NARRAGANSETT CHIMNEY.





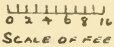
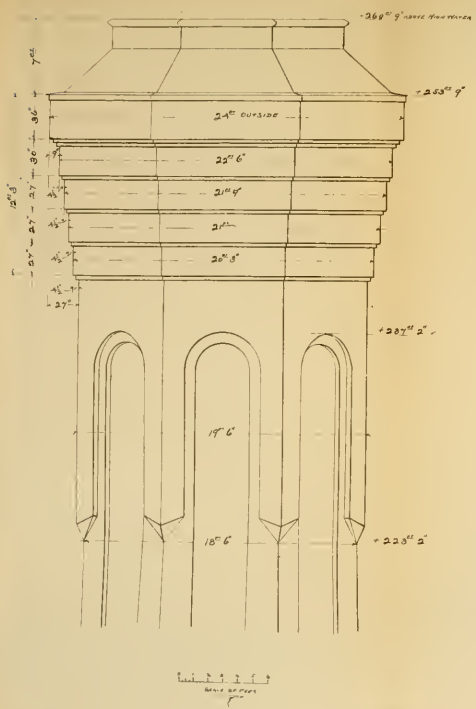
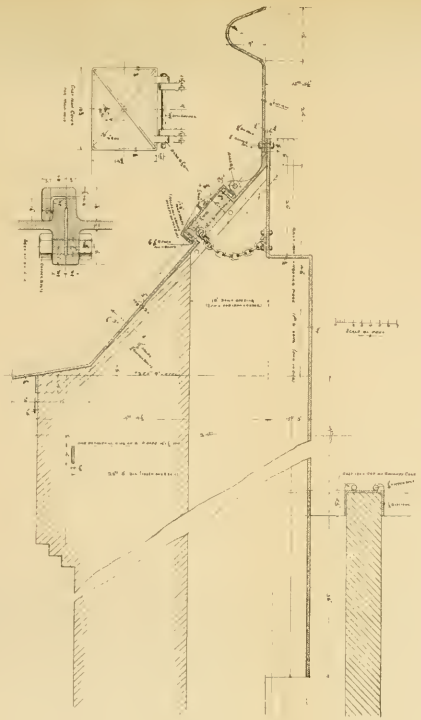
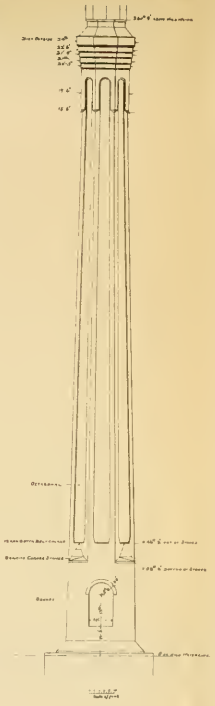


PLATE II
TRANS AM SOC CIV ENG'RS
VOL XXV N°483
HENTHORN ON
NARRAGANSETT CHIMNEY







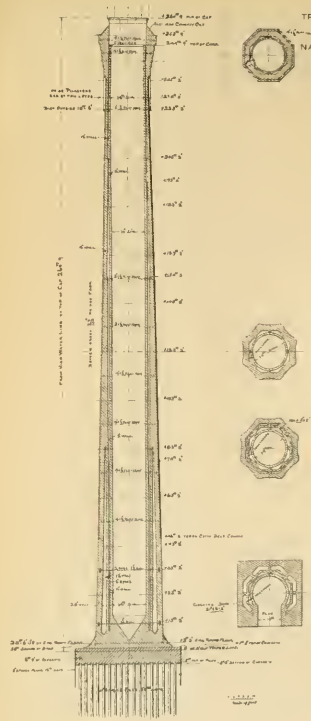
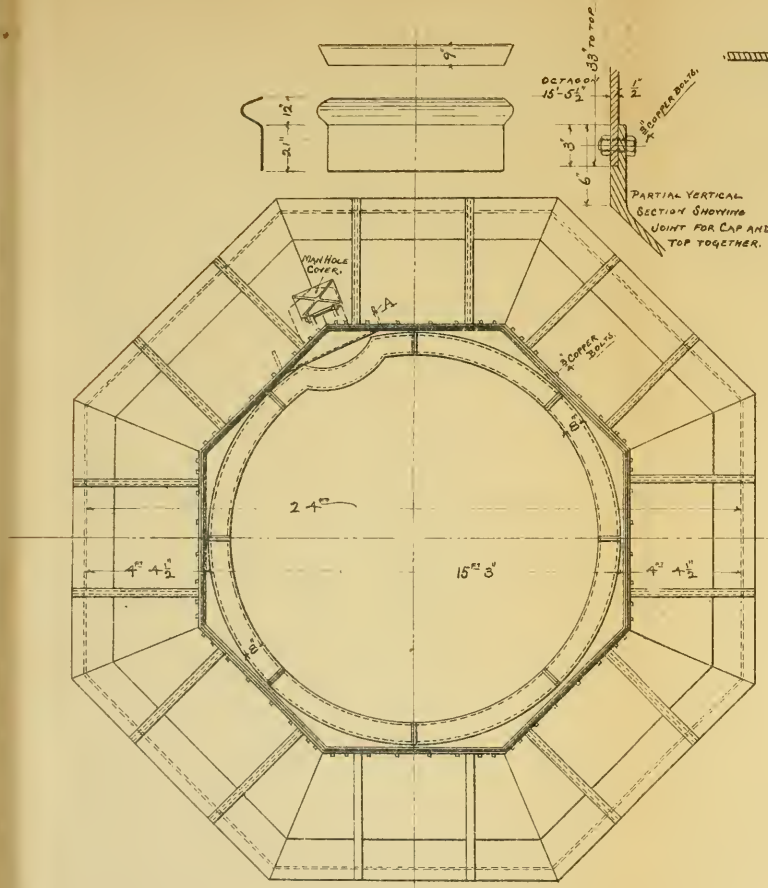
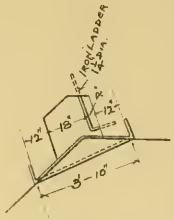


PLATE III
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. No 483
HENTHORN ON
NARRAGANSETT CHIMNEY.

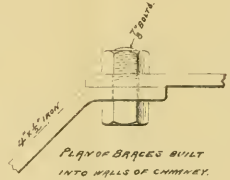


PLAN VIEW OF CAP

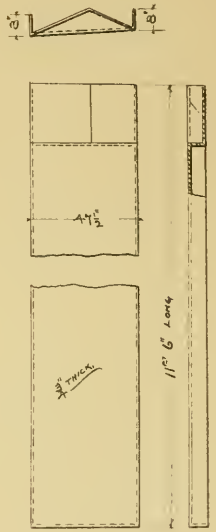
SECTION THRO. CENTER.



PARTIAL PLAN SECTION SHOWING POSITION OF CASTING "A."

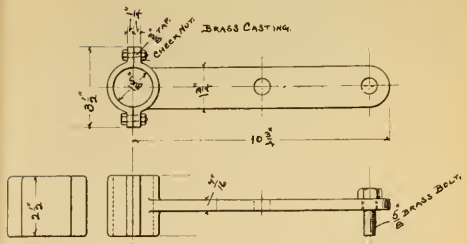


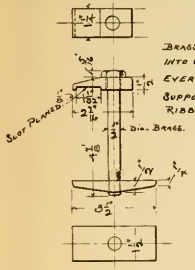
PLAN OF BRACES BUILT INTO WALLS OF CHIMNEY. SCALE, HALF SIZE.

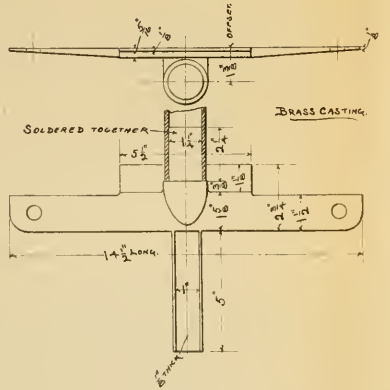


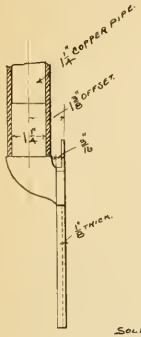
DETAIL OF CASTING A

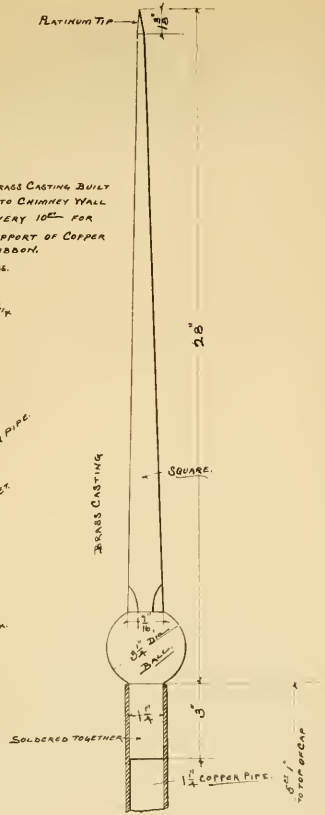
0 1 2 3 4 5 6 7
SCALE OF FEET.

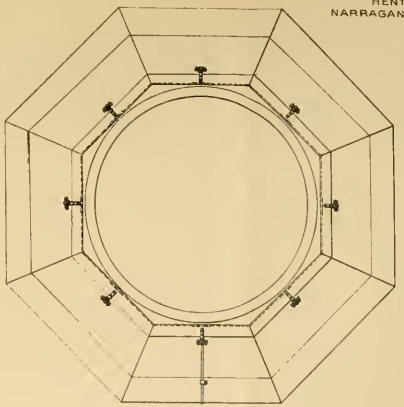


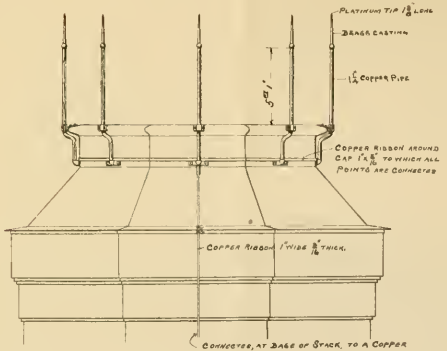








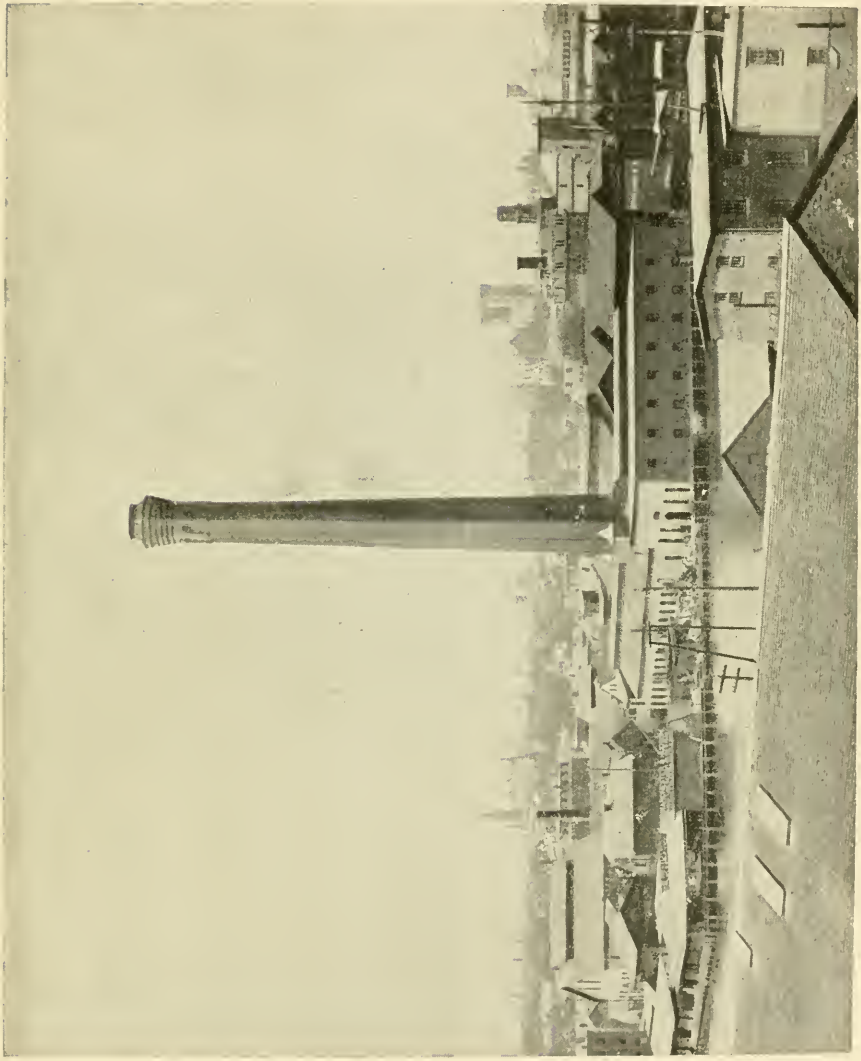




Scale of Inches.

Scale.

PLATE V.
TRANS. AM. SOC. C. E.
VOL. XXV, NO. 483.
HENTHORN ON NARRAGANSETT CHIMNEY.



The amount of material used in the course of construction above the concrete foundation, is as follows:

Brick	1 332 921	
Lime	695	casks.
F. O. N. cement	1 025	"
Portland cement	17	"
Soapstone coloring	99	"
Sand	3 858	"
Cast-iron cap	22 000	pounds.
Cast and wrought-iron	7 215	"
Copper bolts	250	"
Lightning rod and brass castings	326	"

With reference to the drawings which accompany the paper:

Plate I shows a plan of the foundation piling.

Plate II shows an elevation of the chimney, vertical and horizontal sections, an elevation of the head, and section and detail of the cap.

Plate III shows details of cap.

Plate IV shows detail of lightning rod construction.

Plate V shows general view of chimney.

DISCUSSION.

CHARLES E. EMERY, M. Am. Soc. C. E.—I wish to testify to the skill and care shown in the preparation of plans and the execution of this work, and am much obliged to Mr. Henthorn for making a record of the matter in the transactions of the Society. The erection of a ladder between the shell and core is a novelty to me. It causes one to more fully realize that the core is the real chimney and that the remainder of the structure merely gives the core lateral stability. The suggestion arises, why not, in sightly locations, make the exterior portion a tower like that of a standpipe, with such openings as are desirable for architectural effect, and let the chimney proper stand up inside of it? I never have seen any philosophical reason for stopping the lining below the level of the outer walls. It would seem that the cap could be much better protected if not exposed to the action of the gases, and that the draft would be a trifle better if the flue were the higher.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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(Vol. XXV.—July, 1891.)

THE FIRST TRIP THROUGH BIG HORN CAÑON.

By E. GILLETTE, M. Am. Soc. C. E.

So much has been written respecting our large cañons in the West that it seemed as though there was little left to be told. Having occasion last winter to examine the box cañon of Big Horn River, the writer was much surprised to learn that the cañon had as yet been unexplored. It may be safely stated that this is one of the latest, if not the very last of our large cañons to hold out against the explorer. This fact alone has induced the writer to consider his explorations in this cañon worthy of special notice. Through southern Montana and northern Wyoming the cañon has had the reputation of being impassable. Any one who would attempt its passage was considered not in his right mind, and a prospector who ventured part way down the cañon in a boat, was reported in the local press as surely lost. Scouts and hunters who had looked down from the rim of the cañon, had reported a depth of half a mile, with vertical walls, while the river was full of very steep rapids and falls. Taking advantage of the ice which had formed upon the river, the writer was able to make the trip by five days' hard travel.

The Big Horn River, above the head of the cañon, is over three hundred miles long, and drains an area of twenty thousand square miles. The drainage from this extensive section has cut its way through the limestone sag, between the Big Horn Mountains on the east and Pryor Mountains on the west, forming the formidable box cañon of Big Horn River.

On the morning of the 7th of March, 1891, with blankets for bedding, and provisions for five days tied upon a sled constructed of cottonwood poles, the writer started with an old Black Hills prospector, by the name of N. S. Sharpe, on my trip through the box cañon. The head of this cañon is 4 miles south of the crossing of the Montana-Wyoming boundary line, the balance of the 50-mile cañon being situated in the Crow Indian Reservation in Montana. Entering the cañon at the head, we found the river frozen to a depth of 3 feet, but as we journeyed downward, open water was encountered at nearly every bend of the river. The cañon almost immediately from the start rises to a height of 500 feet, with vertical walls. At the foot of these cliffs the talus extends from 25 to 100 feet, being at intervals washed away, leaving the cliff perpendicular to the water's edge. The talus stands at an angle of 45 degrees, and supports a growth of cedar trees which find root among the boulders. The cedar monopolizes most of the cañon, for no other trees grow here save a few cottonwoods at the mouths of side drainages and a scraggy pine or two near the mouth of the cañon.

As we journeyed down the cañon, keeping a sharp lookout for air-holes in the ice and glancing at the vertical walls of limestone on each side of us, we began to realize the fact that we were in the box cañon of the Big Horn River, the terrors of which had been so often repeated us to the measure that "no one had ever gone through the cañon alive." The talus being washed away at the entrance to the cañon as well as at the mouth, "no admittance" stares the pedestrian in the face, no matter from which end he may approach the gorge, and should he succeed in passing these gateways he would soon come to grief at some vertical wall extending to the bottom of the river, while the stream here is making good time down a rapid. This, probably, has prevented the cañon from being more thoroughly explored up to the present time.

Shortly after entering it we passed under overhanging cliffs which appeared to be striving to meet at the top. These cliffs extend over from the perpendicular 70 to 100 feet, and will form a shed, as the

boys said, "for a train of cars on a rainy day." There is ample room under the roof, as the shed towered hundreds of feet above us. Swinging around the bends of the river we soon came to the State line, where we found the balance of our party engaged in running a preliminary line in the cañon to determine its practicability for a railroad route. The men were apparently but a short distance above the river; however, when we had climbed up to them and looked down we realized that a tumble might result disastrously. We found the transit set up on the edge of a cliff with one leg of the tripod nearly parallel to the plumb line, while the transitman was barely able to maintain his position on the narrow shelving rocks; a misstep of an inch would have precipitated him to the hard bowlders 70 feet below. The chainmen made their way around almost vertical cliffs, hanging on with fingers and toes, and as we gazed at them we thought there is no room here for the fellow who usually "coons" a dangerous spot.

Taking observations on the height of the cañon we found the vertical wall to be 600 feet, while a slope of 45 degrees rises beyond, a distance of 200 feet more. This order of the cañon, with gradually increasing height, continues for 2½ miles, when we found ourselves at the entrance of Devil's Cañon, where the walls are 1 000 feet high, as determined by triangulation. As we gazed at the small patch of sky visible, and noted the smallness of the few trees up on the rim, we had a faint conception of the immense work the river has performed in cutting this channel through the mountains. The eastern drainage to the cañon comes from the Big Horn Mountains, while from the opposite side the streams drain the eastern slope of the Pryor Mountains. A peculiar feature of the Big Horn Mountain drainage is that these streams coincide with the divides, cutting their way through what appears the most difficult part of the country, completely reversing the position of the larger drainages in almost any other country. Devil's Cañon carries a stream of water from the new gold camp at Bald Mountain, and this stream, as if competing with the river, has formed a grand cañon of its own. Another feature of the cañon is, that immediately below the mouth of the larger side drains the steepest rapids in the river occur. Tracks of otter, wolves, mountain sheep, and occasionally that of a bear, were noticed in the snow.

As we continued down the cañon the walls decreased in height, until at the Sentinel, 5 miles below Devil's Cañon, they are reduced to a

height of 500 feet. The Sentinel is a pillar of limestone, apparently 100 feet or more in height by 20 feet in diameter, and stands upon a point of rocks, 300 feet above the river. The writer named the rock, as the thought occurred to him that a sentinel might be imagined to be on duty protecting this beautiful cañon from disfigurement by some advertising agent. Had he with "pot and brush" endeavored to paint any of those well-known advertisements seen from any car window in the country, the writer is sure the sentinel would have fired upon him as an enemy to whom no quarter should be given. As night came upon us we went into camp beneath the Sentinel, well knowing we should find him on duty in the morning. From the Sentinel the walls of the cañon vary from 500 to 900 feet in height, vertical, as usual, and apparently no wider apart at top, than high. At intervals small streams break through the side walls of the cañon, forming formidable looking cañons of their own, while from the cliffs above, the smaller drainages, being too weak to cut their way through the limestone pour their waters over the cliffs, forming innumerable waterfalls 500 feet and more in height. At other places what appears a cave in the wall of the cañon is an outlet for some stream, which, having sunk in a recess back from the cañon, has tunneled its way through the wall, forming smaller cascades of from 200 to 300 feet high. Four miles below the Sentinel, the east wall breaks away, widening the cañon a half mile. Here is found a good ford of the river and a trail leading out of the cañon on both sides; this is about the only opportunity we found of getting into and out of the cañon for its entire length. Several good sized streams come into the cañon from the east side, above and below this ford, at the mouths of which gold is found in places, and this appears to be the case all through the cañon, as "colors" were found in every pan of dirt washed out, which had been taken from the bars near the mouth of these streams. At no distant day good placer mines may be found in this comparatively unknown cañon, and quartz mining back in the mountains will receive more attention. A short distance below the ford the cañon closes in again and continues thus for 15 miles, gradually increasing in height as the river approaches the main divide of the Big Horn Mountains. At 15 o'clock of this our second day's trip in the cañon we came to a most noticeable point of rocks, which the writer named, the Tower, from its form and commanding appearance. The west wall of the cañon is here pierced by a side drainage coming in nearly parallel to the main cañon, leaving a knife-edge or wall between

the two streams. This knife-edge is broken off vertically at the end, while a little distance back a square niche has been taken out of the edge 200 feet deep and long, leaving a pinnacle on the point towering 700 feet above the river, with an almost vertical slope on all sides. The Tower is one of the many picturesque points in the cañon, every bend of the river, however, reveals something new, and nothing commonplace is found throughout the entire length of the cañon. Four miles below the Tower we made our second night's stay in the cañon. Travel, so far, had been slow, owing to the depth of snow on the ice, and the numerous portages we were obliged to make around the rapids, where the river was free from ice. Three miles below this camp and 26 miles from the head of the cañon, there is a side drainage coming in from the west which splits a sharp point or angle of the cañon's wall, leaving a shell of rock on either side 500 feet high. This gulch is known by the name of Dry Hollow, and gold is found here in paying quantities. In several bars along the cañon and in places formerly the bed of the river, Sharpe panned out enough gold to show that the "diggings would pay."

Three miles below Dry Hollow Creek a sharp bend of the river forms a high sharp point of rocks, bearing down upon us from a height of 1 000 feet. This point, looking as formidable as the bow of an approaching steamer, has been named Steamboat Point.

Three miles from here Dry Head Creek, the largest drainage from the west, empties into the Big Horn, after having cut its way through the rocks and forming a cañon impassable for 6 miles back from the river. The walls of the cañon continue to grow higher as we work our way down, until at night, forced to camp on a shelf of rock, by an overflow of water on the ice, we gazed up at a cliff 1 200 feet high, and observed by the whirling snow up on the rim that a heavy wind storm was in operation, while there was a dead calm in the cañon. Starting out the next morning at 7 o'clock, we soon passed the mouth of Elk Creek, the third largest stream coming in from the Big Horn Mountains, and here we found immediately below the mouth of this creek a nice level tract of some 80 acres in extent. It is the only smooth ground in the cañon. As we approached Elk Creek the cañon began to widen out, and, rising in terraces, reached an altitude of over 1 000 feet. Here and there ragged points jut out into the cañon, capped with towers and pinnacles, and forming a castellated structure of surpassing grandeur and beauty for the next 10 miles, while the river is cutting its way through the divide. This divide

is passed 5 miles below Elk Creek. The lower part of the cañon here consists of a rough rolling country for 600 feet, covered with cedar and a few pines, then rising in broken terraces to the bare divide, more than a half mile above us. Startling a large band of mountain sheep, we watched these animals speeding along the terraces, jumping down short vertical cliffs in what appeared a most reckless disregard for the safety of life or limb, and they finally disappeared up one of the steep side cañons on a well-known trail.

Two miles below the divide, Black Cañon forces its way to the Big Horn, and duplicates the cañon of that stream as far as the eye can see. Just below the mouth of Black Cañon we came to the steepest rapid on the river, where the water at the present time is confined to a narrow bed less than 40 feet wide, the usual width being from 200 to 300 feet. The river continues narrow from here to the mouth of the cañon, and rapids are found at every bend. Innumerable warm springs pour their waters into the river, and we are forced, for want of ice, to abandon our sled and make the balance of the distance down the cañon, along the side of the stream.

We camped one more night in the cañon, among huge drifts of wood, 25 feet above the river, and can form a slight idea of how the cañon appears during the season of high water. The walls continue to maintain their lofty height down close to the mouth of the cañon. After a hard day's scramble over bowlders and cliffs we passed through the lower gateway at 16 o'clock, dragging our camp outfit behind us, as the ice here has formed sufficiently to barely sustain our weight. We have had enough cliff-climbing for the time being, and are willing to take some chance of breaking through the ice to save another laborious portage. Coming up on to the broad and fertile valley of the Big Horn, it was with a feeling of freedom and satisfaction we made our way down the valley and camp opposite the ruins of old Fort C. F. Smith. Having been confined for five days between those vertical walls of rock, we had no desire to make the return trip up the cañon, especially as of late the weather had been quite warm, and floating cakes of ice showed us that the river was "breaking up." This signified much to us, as it meant longer and more frequent portages. Congratulating ourselves that we at least had "gone through the cañon alive," we started back for camp, over the mountains, having first secured a little flour and bacon to replenish our now empty mess box.

We took the old Sioux trail going back; there was no fear of missing this route as the Indians have piled up thousands of tons of rocks to mark its course. This trail which connects the Valley of the Big Horn in Montana with that in Wyoming has been used for centuries by the buffalo and Indian to avoid the big cañon of the river. No Indian cared to venture into its depths and so this beautiful and comparatively unknown cañon has slumbered on for ages, awaiting the coming of the railroad for which it has cut a deep pass through the mountains, giving an easy grade for the transportation of the products of the country, besides revealing to the traveler probably the finest specimen of a box cañon in the world. We enjoyed the scenery in an almost enthusiastic manner but could not help thinking that it would appear just as grand and beautiful from the rear platform of a Pullman, or from an observation car, as it did from amongst the boulders with a heavy pack upon our back. After three days' wading through the snow we arrived in camp, in good condition to appreciate once more the blessings of full rations and a liberal supply of blankets.

In conclusion, the writer can state that the difficulty and danger of going through this cañon has been, as is generally the case in similar instances, very much exaggerated. When the river is high it would be difficult and unsafe to go through the cañon. Probably the best time of the year to make the trip would be in the autumn, when the water is reasonably low and boats can be used. There are no falls in the river, and but few very steep rapids. He would prefer making the trip in a boat, as in the winter, the ice being quite thin in places, one is liable to break through, when the swift current would take him quickly under the ice. We had an illustration of this above the head of the cañon. A settler was leading two horses across the river on the ice, which suddenly giving way, precipitated both animals into the stream; the current quickly carried them away.

In a subsequent paper will be given a statement of the cost of constructing a railroad line through the cañon with the maximum grade and curve required, accompanying which will be a map and a few views of the prominent features and interesting points in the cañon.

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485.

(Vol. XXV.—July, 1891.)

DISCUSSION OF PAPER No. 444, "REDUCING INTERNAL WASTES OF THE STEAM-ENGINE."

By ROBERT H. THURSTON, M. Am. Soc. C. E.

I am able to add something to the collection of facts and data bearing upon this subject, as presented to the Society in the paper under consideration.

It will be remembered that the writer had endeavored to find a practicable way of reducing the heat-conductivity and the heat-storing power of the internal surfaces of the steam-engine cylinder, as had many before him, from the time of Smeaton and Watt, and he believed that he discovered that it would be possible to form upon these surfaces a non-conducting stratum, integral with the material of which they were composed, with the metal of the cylinder itself, that should intercept the heat tending to pass from steam to metal and to return from metal to steam after its temporary storage, in this manner, finally to be wasted as heat, and without transformation in any appreciable degree into work. This he had proposed to bring about by first, in any convenient and inexpensive way, effecting a partial solution of the superficial portion of the metal, leaving a spongy layer, consisting, as in the familiar case of "transformation into carbon," as it has been vulgarly supposed to be, of the interior of condensers and of channel-ways in condensing engines through the slow but extensive action of the passing stream of air-charged and hot water, from condenser to air-pump; then, by the im-

pregnation of this sponge with a resin or other non-conducting material produced by the painting of this surface with drying oil, or with any available substance that should thus be found able to permeate and cling to the porous surface.

In the paper presented to the Society it was stated that experiment had shown it to be practicable to reduce the conductivity of the surface to an important extent by simply adopting the first of these two processes; then it was shown that the second produced a still greater effect. Those experiments were laboratory experiments, made to determine whether the idea of the writer was correct, and whether the physical process and its results would be as anticipated.

This last question has been fully settled. It is found that simply oiling a surface capable of giving a good foundation and holding-power for the grease, will give a reduction of 10 per cent. in the heat-wasting power of the iron; that a very moderate degree of solution of the surface will, without other action, give a reduction of 20 to 40 per cent.; and we now find that the combination of the two processes described in the paper under discussion, gives a gain of 60 to 70 per cent., accordingly as it is more or less complete. We do not despair of securing so effective and so complete an action, in this way, as to reduce the heat-storing and wasting power of the metal of the cylinder to an insensible amount. It will then only be necessary to secure dry or moderately superheated steam to bring the efficiency of the real engine up approximately, perhaps sensibly, to that of the ideal engine; and thus reduce the consumption of steam and of fuel to substantially the figures to-day considered as purely hypothetical, and much below anything actually attained in the best of modern practice.

Experiments recently made on a small stationary engine (carefully adjusted until it did the best work in its history, and far better than when first set up) have shown a gain, in that case, of over 10 per cent. in expenditure of steam; though the treatment was superficial and only affected a depth measured in ten thousandths of an inch. These results have been duly reported to the A. S. M. E., by Mr. Royse, who carried out the investigation at my suggestion, and it is proposed to continue it on a larger scale. It may be presumed that, like the action of the steam-jacket, the effect of this treatment will be very variable with the size, character and conditions of operation of the engine to which it is applied.

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486.

(Vol. XXV.—July, 1891.)

DISCUSSION ON PAPER No. 465, PERMANENT EFFECTS OF STRAIN IN METALS.

By WILLIAM METCALF, M. Am. Soc. C. E.

Professor Thurston, in his reply to my discussion on this subject, unintentionally, no doubt, mistakes my remarks. He says (see Vol. XXIV, p. 186): "The brooming and splitting of bars, to which he refers," etc., etc.

I made no allusion whatever to the "brooming and splitting of bars," such bars could never be shipped to any person, nor could bars with the black core I mentioned be sent to any one safely, for their sure failure would lead to their return to the maker, as I learned to my cost before I discovered the nature and the cause of the trouble.

The weakness, brittleness, tendency to split, etc., that I mentioned, occur long before a black core is produced, and still much further away from "brooming and splitting."

As one illustration, take the section of a handy desk rule and paper cutter combined, which is much in use, and is made of thin steel. The center is bent to nearly a full circle, and from this the metal is bent sharply outward at each side so as to make the sides flare at about 120

degrees. This is not a very difficult bend for unstrained metal to endure. It is made a little over 12 inches long, and is bent up from a strip of cold rolled steel. No one who has not tried it can realize the difficulty there is in taking the finest and most ductile dead soft metal of not over 0.10 or up to 0.20 carbon, and rolling it barely enough to have a smooth surface, and yet have it endure that bend lengthwise of the bar. I could multiply instances by the hundreds if it were necessary. At the last we declined to make any cold rolled steel to endure a double bend, or a lengthwise bend except at a very high price, which was necessary to cover excessive cost.

It is quite common for people to order hundreds upon hundreds of short strips, say 2 to 3 inches wide, cut lengthwise across the grain from hot rolled sheets, so that when they are cut again into little strips, they will have the grain running lengthwise of the final strips. In many cases it increases the cost largely to cut sheets in this way, and I have suggested cheaper modes, only to be told that they knew what they were about and were willing to pay for what they wanted. As a rule, our shrewd manufacturers do not spend their money on mere fancies.

Yet we are told, "If a metal be subjected to a stress of any given kind, or in any stated sense," sufficient to produce permanent strain and set, then its ultimate resistance to that, or to *any other kind of stress*, will be sensibly increased, and in *all directions*, whatever the line of the deforming stress." (The italics are mine.)

The statement is not accurate, it does not cover all of the facts, and I regard it as dangerous.

In regard to Professor Johnson's remarks about the modulus, he clearly, did not catch my meaning, which was that the formula used for calculating springs based on the modulus of elasticity showed that any of the bars I mentioned, having the same modulus of elasticity, would make springs of equal capacity, whether tempered, untempered, annealed or not annealed—a manifest absurdity.

The bars were in three sets, consisting of low, medium, and high spring steel; they were all of the same size but treated differently, and yet all gave practically the same modulus of elasticity. The calculation was not mine, nor the conclusion, but eminent engineers have told me that the theory was right, only it does not apply to springs.

R. H. Thurston, M. Am. Soc. C. E.—I have no doubt that I misinterpreted Mr. Metcalf's remarks about the splitting of his steel. On that

point he is an authority, at any rate, and I should not think of questioning what he has to say. As he now puts it, I see more clearly what he intends, and know that he is correct. My own statement, "If a metal be subjected to a stress in any stated direction, its resistance to stresses in other directions will be sensibly increased and in all directions," was exactly what I intended to say, and is precisely accurate in the sense in which it was intended that the words should be interpreted. I did not intend to say, and did not say, that if the metal were overstrained this would be true. We all know that a metal can be spoiled by overstrain, especially, in such cases as are referred to by Mr. Metcalf. My intended proposition is both justified and proved by the experiments described in that paper, and, I think, needs no further explanation. In those experiments, stress was applied in every principal direction, exceeding the elastic limit, but not going to the limit of incipient destruction, and, in every case the proposition was justified—as it has been by a thousand other similar experiments before and since the paper was presented, and as it always will be, I imagine, to the end of time. I do not think that it will be generally misunderstood. Stresses well inside the limit of rupture were, of course, intended and specified. In practice, they rarely much exceed the elastic limit; in the examples given in illustration, they were just above that limit.

J. B. Johnson, M. Am. Soc. C. E.—Mr. Metcalf seems to be laboring under the mistaken impression that theory indicates that the capacity of a steel spring is wholly a function of the modulus of electricity of the material. In his former discussion, he said, "I have a great respect for the modulus of elasticity, because since I worked it out as a student in Troy, I never understood its application in formulas involving all manner of strains." This statement is a sufficient explanation of the entire discussion on this point. If he had understood its application he never would have made his subsequent statements in regard to the failure in its application to steel springs.

The fact is there is probably no single function used in engineering formulas concerning which there is a closer agreement between theory and practice than in the case of the modulus of elasticity. It is now known that this modulus for all grades of wrought-iron and rolled steel, of all degrees of hardness, and of all stages of tempering, probably varies between 27 000 000 and 30 000 000. Any results obtained outside these limits are more likely to be erroneous than that the modulus

really has such values. The fact that poor observations have given us, even in standard publications, erroneous values outside these limits, is not to be placed to the discredit of the theoretical formula. But this is not quite Mr. Metcalf's trouble. He argues that because the modulus of elasticity was found to be the same for all grades and tempers of iron and steel, therefore they should theoretically have the same capacities when shaped into springs of equal volume. Of course persons who do understand the modulus of elasticity would not draw any such conclusion, and it is difficult to conceive that there are many "eminent engineers" in this country who would state that the theory of elasticity "does not apply to springs" within their elastic limits.

If a plain statement of some very elementary principles may be pardoned in this connection, it may be as well to say that the modulus of elasticity is simply a ratio between any given distortion (within the elastic limit) and the stress produced by it. It is an indication of the stiffness of the body. The higher this modulus, or ratio, the stiffer the material. Since wrought-iron has the same modulus as spring steel it has the same stiffness. That is, it will require the same load to deflect two similar bars of these materials the same amount, this amount being less than their elastic limits. This is not only a theoretical fact, it is constantly proved in practice. In fact the identity of their moduli of elasticity is only known from just such tests; and within the elastic limit of the iron, one bar is as good a spring as the other, as I took occasion to say, in my former discussion. So much for the stiffness of the two bars.

When we speak of the capacity of a spring, however, we may mean either the greatest static load it will carry and fully recover, or we may mean the greatest shock it will resist (absorb) without permanent set. In the former case, where strength only is involved, it is no function of the modulus of elasticity at all, but only of the maximum fibre stress (strength) which the material can resist without permanent set. In the latter case, where energy is to be absorbed, the capacity of the spring to do work (in foot-pounds) is a function of both the modulus of elasticity and of its elastic limit strength. Given two materials having the same moduli of elasticity, their capacities to absorb energy, or do work, will always vary, for all kinds and shapes of springs, directly as the square of the elastic limits of the materials. Thus, if a wrought-iron spring has an elastic limit of 30 000 pounds

per square inch, and a highly tempered steel spring has an elastic limit of 90 000 pounds per square inch, then the steel spring will have a capacity for resisting shocks of just nine times that of the wrought-iron spring of the same size and shape. There is nothing new or strange about this. Many elementary works on the mechanics of materials give all these facts, and the exact formula to use for any kind of spring. I would undertake to compute the maximum load, shock, or deflection of any standard style of steel spring from the theoretical formulas and obtain results very closely agreeing with any actual tests which might be put upon them. There is an impression abroad that there is an eternal conflict between theory and practice even in engineering. This I always deny. Any theory worthy of application is simply a generalization from actual experiment, or, it is generalized practice. Anything, therefore, coming from a high authority, which unjustly tends to deepen the prevalent distrust of wholesome theory in engineering, should, in my opinion, be immediately corrected, and nullified if possible.

Mr. METCALF.—Prof. Thurston's letter so modifies his paper that I think all will be clear now, he still exaggerates my position slightly, but that is no matter. I regret more than ever that he was not at Cresson to discuss his paper, then everything would have been cleared up more thoroughly and in a nicer way. It is a pity that we cannot always have good face-to-face discussions.

Replying to Prof. JOHNSON.—There is some danger that this discussion will become funny, or wearisome, and altogether unprofitable.

I repeat, I do not see how a modulus of elasticity obtained by stretching a piece slowly in tension, is going to furnish data for the computation of all sorts of shocks and vibrations applied all at once, producing torsion, extension, compression, deflection, and vibrations all in the same instant; this is what happens in springs. I may be wrong, I confess I never tried the computation, simply from want of faith; the party who did try it failed ingloriously; if Prof. Johnson can calculate the strength of a helical spring accurately, not approximately, he can teach the world something. I know the formulas, and I know about how closely they work. Professor Johnson says, "where strength only is involved it is no function of the modulus of elasticity at all." I agree with him entirely. I have made springs to other people's formulas involving the modulus of elasticity as an indicator of strength and I know just how near they were.

I am sorry Professor Johnson thinks I wish to belittle theory to exalt practice; I think his conclusion is hardly just, and I think if we could talk together ten minutes we would not find ourselves differing greatly. My whole objection is to the drawing of general conclusions, or laying down general laws, upon insufficient data; and since Prof. Thurston has modified the statement that led to this discussion, by saying, in all "principal directions," the addition of the word "principal" ends the discussion, as each person may give his own definition to the word principal, and the modification makes the statement no longer a general law. Finally, I hope Prof. Johnson will allow some weight to intelligent experience even when it is not backed by testing machine data, and that he will not confuse it with that ignorant practice which knows not its own ignorance, the most deplorable of all mental conditions.

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(Vol. XXV.—July, 1891.)

SECTIONS AND MECHANICAL CONDITIONS OF CAR WHEELS.

By P. H. GRIFFIN, Assoc. Am. Soc. C. E.

It is strange that the mechanical conditions under which car wheels are used have received so little attention from engineers and railroad men generally when the great attention given permanent way is considered. A vast sum has been expended in the past ten years in improvements of every kind calculated to make perfect road-bed. The subject has received more attention from engineers than any other connected with their work. Inducements of every kind have been held out to the men responsible for the maintenance of proper conditions. Inspections are regularly and constantly made. In fact the subject as a whole is well understood, and the possibilities and defects are thoroughly known. Ingenious appliances have been brought into use for detecting bad conditions, and the track that is not in nearly perfect condition to-day is so because of a lack of care, or from necessity not allowing the carrying out of work that is not questioned as to its value.

Almost equal attention has been given the matter of equipment. Heavier and more perfect construction has been followed, and everything that the best mechanical practice could suggest has been carried

out with a view to making cars and locomotives as well adapted as possible to the service in which they are used.

We have then, the highest types of mechanical construction in the the two great divisions of practical railroad work—permanent way and equipment. Between the two comes the car wheel; the one thing that must necessarily receive and transfer all of the load and power; the one active agent that enables motion, and supplies through mechanism operating upon it the only means of control over the enormous forces developed and used.

It would hardly be considered good mechanical practice to build a costly and perfect stationary engine and connect it with a heavy piece of machinery, also costly and perfect, devoting all possible care to both objects and little or none to the medium through which the power was transferred from one to the other. Yet in such a case all that could be considered would be loss of power and danger of accident due to imperfect conditions in the connecting parts. It would be only natural to expect that the care bestowed on the first two items, costing, perhaps, one hundred times the third, would be proportionately given to the last one, and that the order of mechanical perfection would be equal in each part. Consider again, a case where the third part was possibly neglected and that every defect in it was multiplied thousands, yes, millions of times, with a direct result on the other two parts; that would be, as stated, a strange case; yet it is precisely what exists on all of our railroads to-day. What engineer can describe the section of car wheel best adapted to safety? (We are now speaking of chilled wheels; the kind that is under 90 per cent. of the equipment in this country.) Who can say from actual knowledge how much the wheels under the cars of his company vary in diameter from each other on an average; or how much they are out of round or out of balance, or whether wheels of the same weight are placed on each axle?

It may be answered that of course such things must be right, or that it is not the province of the engineer to investigate such subjects; or again, that a small variation in these particulars is not important. Considered as a whole, good management in practical matters of the kind will embrace all conditions affecting every branch; if one is neglected, the results that might be obtained from the others are lost. Therefore, the subject is one in which a proper interest may be taken by all concerned in the construction, maintenance or operation of railroads.

As stated, 90 per cent. of the equipment of the country has chilled car wheels under it, and therefore the subject is determined mainly by wheels of that class. Of the comparative value of different types of wheels a few words will be said later on.

The matter will be presented under two heads:

First.—Proper Section and Methods of Manufacture.

Second.—Mechanical Defects, their Causes and Results.

The best section of wheel depends largely on the service intended and upon the quality and character of the wheel, but certain lines should be followed irrespective of these two conditions on all steam roads.

The strains imposed on a wheel are of two kinds; the first consequent on load carried and speed attained; the second that which results from the use of brakes. The first strain multiplies the second in a definite degree. Given a speed of 20 miles per hour with a load of 5 tons per wheel, and a pressure of 50 pounds in a 10-inch Westinghouse cylinder giving 10 000 pounds pressure on each brake shoe. (You may note the diagram showing levers and pressures in Plate II.) One-half of this pressure applied for five minutes under the conditions named will develop through friction a certain amount of heat; the wheel must take up the heat and consequently expand. Just what strain is developed in the wheel during the operation is not easily determined, but it is over 100 tons. You will be able to compute the increase in strain at higher speeds and with greater loads or longer application.

As to extreme limits, they must be considered. On heavy grades it is common practice to carry double the load named, at double the speed and with double the continuous brake application; presumably each condition would double the strain. The last one, *i. e.*, heat developed by friction would increase it in a greater ratio because it would be a continually increasing condition.

It is not only necessary to consider in connection with this matter the speed proposed for general use or the loads intended to be carried, but it is absolutely necessary to consider the maximum that could possibly be imposed, and to provide a margin of safety beyond that. You provide for strains in other construction that are beyond anything met with in service. The same must be done in a car wheel if absolute safety is sought. You must also remember that in most construction only conditions under your own control are provided for. Suppose you were sending your bridges all over the country to be used by all sorts of care-

ful and careless people; or that you were taking all sorts of bridges and using them without particular investigation of their quality. That is impossible of course, in practice, but that is actually what is done with the car wheel, and if a man was given the choice of taking his chances for life and damage over a broken wheel or defective bridge, there would not be much choice. There have been more people killed and property destroyed with broken wheels than by defective bridges. So much for the need of a known basis to follow in the construction of wheels.

Plate I illustrates the New York Car Wheel Works' standard section of 33-inch 600-pound car wheel. As this style of section is followed in different weights and thickness of metal in all diameters, the explanation will apply to them all. It is, as you see, the regular double plate style of wheel known as the "Washburn" pattern. The section lines and curves, however, are peculiar to the standard named, and are based upon many years of practical and extended experience, as well as upon long continued and elaborate investigations into every feature composing the whole. A 33-inch wheel of this section with $\frac{7}{8}$ths-inch plates and of standard hub, bracket and tread dimensions, weighs 550 pounds, and is the minimum in this diameter for steam railroad service consistent with safety. Wheels are made to this standard in all diameters from 24 to 43 inches, and weighing 400 to 1 200 pounds.

In the figure shown, the hub section is calculated to stand a strain of 150 tons in pressing on the axle; this is more than double the pressure used. Wheels of heavier section are made to higher standards in this respect, the limit going up to 500 tons. The core dividing the double plate is made, as you will note, with a slight lip on the outer edge. It is common practice to bring the lines of the core at this point to an abrupt finish with the idea of making the metal at the junction of the plates heavier, and therefore presumably stronger. As a matter of fact, the opposite result is produced when this is done, for the greater body of metal remains fluid and soft a little longer at that point, and therefore, the shrinkage of surrounding parts is fed from it, leaving the metal porous and weak at the very place where it should have the most strength. To this cause is largely due the fine cracks found at the junction of the plates when wheels are subjected to continuous heat service. These cracks do not always go through the plates, often do not penetrate over $\frac{1}{8}$th of an inch, but they form starting points for more serious defects.

The core dividing the two plates is known as a "pan-core," because it is made in a dish or core-pan formed to the shape of the bottom of the core; the top is made with an iron sweep pivoted on a center hub in the core-pan. As per Plate II, "*a*" is the pan, "*b*" the hub, "*c*" the sweep; "*d*" indicates a ring set in the core-pan and used with a larger hub to give extra metal when a larger sized bore is wanted in the wheel. The use of these rings is very necessary, otherwise wheels are bored out for large axles with the consequence of leaving insufficient metal in the hub to withstand the strain. There should never be less than $\frac{7}{8}$ths of an inch of metal in the wheel hub at the center of wheel seat—the thinnest point, after it is bored for the axle. The core is sustained in position on the mould by the legs "*e*" fitting into depressions in the wheel mould made by prints on the pattern.

"*F*" is a depression in the top of the core to receive the end of the chaplet which keeps the core in position. Cores are frequently used without this last provision, but it is very necessary, as it provides an extra quantity of metal in the form of a boss on the inside plate of the wheel, at a point where the iron chaplet is very liable to cool the hot metal quickly, and chill it for a space of $\frac{1}{2}$ an inch or more around the chaplet, thus providing a starting point for defects. You will note that where the core-leg leaves a hole in the bottom plate, the latter is re-enforced by a fillet; this is very advantageous for the same reason as that given in the case of chaplets.

A great deal depends on the use of perfect pan-cores and upon the exercise of great care in every detail concerning them. When the best attention is not given, internal defects result in the wheel, very difficult to detect. If the chaplets are not exactly and firmly adjusted, one plate will be thinner than the other, and the metal consequently harder and weaker.

The bracket is $1\frac{1}{8}$ inches thick at center and 2 inches deep. It fulfills two functions; the first to act as a runner or gate for the molten metal flowing into the tread, and the second to afford strength and support to the tread and flange in service. The section should therefore be determined by the area necessary to admit of the hot metal passing freely and rapidly to the flange and tread, and also by the dimensions best calculated to meet the expansion and contraction resulting from brake service. The bracket, as you will note, is curved to the radius of $5\frac{3}{4}$ inches inside, and $6\frac{7}{8}$ inches outside. This is a very important feature.

As the hot metal is poured into the mould, brackets of this shape allow it to pass to the tread of the wheel with the least possible strain upon the casting, at a time when it has little or no practical consistency. It is vitally important that the metal be poured into the mould quickly and as hot and fluid as possible; this because of better homogeneity and soundness in the casting. In regular practice the metal should be cast into a 600-pound wheel in twelve seconds. As it takes with runners, head, screws, etc., over 650 pounds of metal to cast a wheel of this weight, you judge of the effect of practically dumping almost instantly nearly one-third of a ton of molten metal into a sand mould. Things have to be pretty nicely and perfectly adjusted to admit of that, and the circular bracket is one of the most important features in making it possible. The style of bracket generally in use is the ogee, or double curve. Presumably this style gives a better support to the tread, running at right angles to it. In practice, however, it is objectionable, for two reasons:

First.—In casting the wheel, the metal forming the tread flows through the brackets and strikes the cold chill forming the tread surface. It is thrown against the chill at a right angle, and must separate and pass to the right and left with many particles partially chilled or frozen, and with no chance for them to remelt and become homogeneous with the balance of the casting. It is almost impossible to pour metal as rapidly through brackets of this kind as through the single curved ones, because of the strain upon the casting when the mould is filling, or, to pour as hot metal, for the same reason. With the single curved bracket the pressure on the mould is not direct on its exterior; it is lateral all around.

Second.—In after service when the brakes are applied and the wheel expanded, the single curved bracket will adjust itself to the expansion and contraction better than the double curved bracket, and, consequently, prevent what is known as "cracked brackets."

The tread section shown has been considered by some too light. This is probably because of comparison with other patterns having heavy section at this point, to give an apparent solidity to the wheel. The uniform distribution of metal in the wheel is all important, and no greater mistake can be made than to add to parts which can be judged as to thickness and weight, at the sacrifice of parts the section of which cannot readily be ascertained. It is all important that the part of the wheel directly under the action of the brake-shoe be not of extra

thickness at the sacrifice of metal in the plates and brackets. All the strain is thrown on the latter and the greater the bulk of metal heated in the tread the more liable the wheel is to crack and break. Due consideration should be given these facts, and proportions followed that will provide against danger from every source.

The point of junction of single plate to tread is very important, chiefly on account of the effect of expansion from brake service. The contact of the brake-shoe is necessarily with the flat surface of the tread, and the metal in the flange acts as a preventive against expansion. When the single plate joins the tread too near the top of the latter, it forms in connection with the flange and brackets, a complete resistance to the expansion of the wheel. If it is kept lower down, when the tread is heated the top portion can expand and the whole casting can, if properly constructed, give a little without fracture; but when, as is sometimes done, the single plate is brought up near the top of the tread, there is, as stated, complete resistance at every point. That is not the way to prevent breakage. The old adage of the "bough that will bend, etc.," applies here with force. The single plate is sometimes brought near the top of the tread to prevent the latter "chipping" or breaking out, but trouble of this kind does not occur if wheels are properly and carefully made, and the practice is dangerous for reasons stated. Any style of chilled wheel can be tested for capacity to stand expansion from brake service by the following simple means: Lay it horizontally on the foundry floor; provide a sectional pattern that will allow sand to be moulded around the wheel and leave an opening ½ an inch wide all around extending from the throat to the edge of the tread. This may be quickly filled with molten iron from several small ladles, and the result is a band of hot metal ½ an inch thick and 4 inches wide, cast around the wheel. The effect is the same as that produced by a sudden and severe application of brakes. If properly constructed, the wheel should not crack in the plates with the application of the test. It causes a series of fine vertical cracks around the wheel and renders it unfit for service, but it may be used with entire satisfaction as to the value of results on wheels condemned for slight foundry defects. So much for the general details of construction.

It does not follow at all that good wheels will be made because a pattern of proper section is used. That is the first necessity; the second is the method by which the wheels are made. The manufacture of car

wheels is hard, laborious work. A first class moulder, if given a car wheel to make with ordinary foundry tools and appliances, would make possibly one or two for a day's work, and consider it a big day's work at that. Special tools and appliances are provided for the business, however, and one man with a helper will turn out on the average eighteen wheels per day.

The work is done almost invariably by the piece and is commenced and finished in ten hours or less. Half of this is given to moulding and the balance to casting. To prepare and finish eighteen moulds in five hours necessitates doing the work at the rate of one in less than twenty minutes. This does not afford much time for the proper carrying out of the many necessary and important details, and with workmen not very expert, many chances are taken for imperfect work.

The mould having been prepared, however, presumably in a proper manner, the quality of metal poured into it determines vitally the character of the wheel. On the preparation of the metal a few words may be said. The cupola is prepared and charged. If the metal is of good quality and strength and is properly poured into the mould all is well so far. But the most exacting attention to every detail is necessary in preparing and melting the iron. If not given, it may not always produce dangerous conditions, but it will not produce perfect ones. A chill test block 2 inches square by 6 inches long cast from the metal should have a chilled surface at least $\frac{7}{8}$ of an inch deep; this degree of hardness on the test will give $\frac{1}{2}$ to $\frac{5}{8}$ of an inch chilled surface on the wheels. A test bar 1 inch square by 12 inches long cast from the mixture should not break at less than 2 500 pounds when supported at the ends and loaded at the center; it should carry a load up to 3 000 pounds.

It is the practice of some wheel-makers to establish their results in this way by deductions from bars of different section and length, but on account of the decrease in strength due to extra hardness of small sections, deductions obtained in this way are unreliable. It is also stated by some that proper strength cannot be obtained in a bar of 1-inch section on account of hardness of metal. This is an excellent proof of the value of the 1-inch section when it is considered that the greater part of a wheel is less than 1 inch thick. If a proper degree of strength cannot be obtained in 1-inch bars how is it to be obtained in plates of less thickness in the wheels? It looks well, of course, to see great strength indicated on test bars of heavy section, but satisfaction as to

results obtained in that way is not all that could be desired. Any wheel-maker who cannot furnish test bars from his mixture, 1 inch square, that will carry 2 500 pounds load between supporters 12 inches apart, is not using a mixture that is what it should be; and if such bars will not carry 2 000 pounds the wheels are positively dangerous for use.

After the wheel is cast it is placed in the annealing pit. Properly speaking, car wheels are not annealed; they are slowly cooled for the reason that in the process of manufacture the outer part of the tread is cooled and set at a degree of heat lower than that existing in the body of the casting (this on account of the chilling process), and the entire casting must again be brought to a uniform heat and cooled evenly. The cooling pits as they may be properly called, should be in dry ground. If dampness is found and steam is seen arising from the pits while the wheels are cooling or when they are being removed, shrinkage strains will certainly be found in the wheels and they will be liable to break in service. When such conditions exist they are always indicated by a reddish color on the wheels when cold.

We will now take up the second division of the subject: Mechanical Defects, their Causes and Results.

What engineer would consider for a moment the use of a track in which the following conditions existed? A rise or buckle of $\frac{1}{16}$ of an inch, and a curve in and out of $\frac{1}{4}$ of an inch in every 9 feet. Yet, as a mathematical proposition, what is the difference between running two wheels of exact diameter and pefect rotundity, fitted to an axle, over such a track as described, as against running two wheels $\frac{1}{8}$ of an inch out of round, and that vary in diameter $\frac{1}{16}$ of an inch, over a perfectly true track. The same rise and fall of the load must occur. The larger wheel would gain $\frac{1}{4}$ of an inch in every 9 feet, and it is manifest that it cannot continue gaining; it must be forced back or the smaller one pulled forward, or the wheels would not stay on the rail. You may think the conditions cited in the latter case are exaggerated, but, in fact, they are the minimum conditions of variation in wheels. The Pennsylvania Railroad specifications now under consideration for adoption by the Master Car Builders' Association accept wheels that do not vary more than $\frac{3}{32}$ of an inch from a true metallic ring placed over them. To place such a ring over a cast surface, not tooled, would certainly take $\frac{1}{64}$ of an inch all around, making up $\frac{4}{32}$ or $\frac{1}{8}$ of an inch. All things considered, to make castings weighing $\frac{1}{4}$ of a ton and over, true to $\frac{1}{16}$ of an inch to center as they come from the foundry, is remarkably good practice.

On the question of variation in diameters, $\frac{1}{16}$ of an inch is a very low average. Not alone the original, but the condition after service must also be considered. To keep a driving belt in the center of a pulley the casting is turned crowning at that point, and the belt stays there under every condition. The tighter it is put on and the more power transmitted, the more difficulty found in running the belt off the center. It is exactly the same with a car wheel. The highest point, or rather the greatest diameter will lead until an excessive condition forces it back to begin the same labor over again. The apparent result of all this is readily seen. Cars sway and jolt as they run. Far less than they did, of course, when the track as well as wheels was imperfect; but still they do it proportionately. The real result is more difficult to appreciate; but considered from a single arithmetical and mechanical standpoint it is evident enough in its magnitude.

The load on a wheel is, say, 5 tons, the speed 30 miles per hour, the variation in diameter from its mate $\frac{1}{16}$ of an inch—diameter 33 inches. The larger wheel will gain three and one-third times $\frac{1}{16}$ of an inch at every revolution. It will revolve six hundred and eleven times and would travel, if it could, 10 feet more in every mile than the smaller one. Of course, it cannot do that; the flange would not permit it; but it gets on somehow, pulling and hauling, grinding and wearing, carrying its heavy load all the time with probably nearly as much power wasted in overcoming defective conditions as would be required to do the real work if proper ones existed.

Again, take the matter of balance. Take the very low estimate of 5 pounds as the amount each wheel is out of balance; I have seen wheels five times that amount out. At 30 miles per hour a 33-inch wheel revolves three hundred and six times per minute. Fifteen hundred and thirty pounds must be swung around the circle described by its circumference every minute on every wheel under each car (eight wheels), therefore 12 240 pounds must be swung around every minute; on thirty cars 367 200 pounds or 184 tons must be moved that often. What power is required to swing 184 tons around a 33-inch circle once a minute for three hundred and twenty minutes—the time it would take such a train to run 100 miles. Remember this calculation has nothing to do with the motion of the train; when that is considered fresh questions arise. What effect does it have on retarding motion?

It may be said that the support of the wheel on the rail counteracts

to some extent lack of balance, but there is ample margin for all possible allowance that way and still have a remainder that is rich in possibilities. The precise nature of these results would be exceedingly difficult to get at by computation; it could only be determined by actual test of perfect conditions as against ordinary ones. The larger wheel unquestionably leads at all times, and the smaller one necessarily draws behind. The greater the load and higher the speed the more certain an extreme condition of this kind will prevail. With the flange grinding the rail instead of running free, power is wasted and undue wear occasioned. It is difficult to locate loss of power or rail wear caused in this way in specific cases, but when the car wheel speaks the whole story is told. Flange wear is the leading cause of wheel failure to-day in every type of wheel, and it has grown in exact proportion to the increase in load and speed.

These are not mythical propositions. We may content ourselves, as the boy did who carried grain in one end of the bag and stones in the other, simply because that had always been the practice. Or we may rely on some unknown law of compensation to make up for such conditions. As a railroad man once told me (not an engineer, I am glad to say), the cars had a certain amount of motion anyway, and these things all equalized themselves when the weight and the work happened to be off one or another particular spot.

There is no mystery about what constitutes good mechanical practice in other things. If you go through your shop and find shafting out of line, pulleys mounted out of center, or vibration in both, due to lack of balance in the pulleys, you stop it at once. It is bad practice—dangerous work. Your shaft revolves perhaps two hundred times per minute—a pretty good speed for shafting. Your pulleys, say 33 inches in diameter, weigh 300 pounds. If they are a couple of pounds out of balance, $\frac{1}{16}$th of an inch out of true; if your line shaft is out of true $\frac{1}{16}$th of an inch in every length, you would not assume the responsibility for that sort of work many minutes and expect safety or economy as a result. But you accept car wheels revolving three hundred to six hundred times a minute, weighing twice as much as the pulley, and from 2 to 20 pounds out of balance, and you do expect safety and economy as a result. You could not mount a pair of wheels taken from under a car in the heaviest bearings and revolve them five hundred times a minute. Leaving all considerations of load carried, tons of brake pressure applied,

and the multitudinous conditions of service, aside, and dealing with the simple fact of running the wheels in fixed bearings at that speed, it could not be done, simply because they are out of balance. You would not stand around at close range when it was being tried, either. But you put them under a passenger car and repeat the same conditions, or greater ones, with little thought of consequences. Centrifugal force and power required to do work under one or another condition have no different cause or effect in either case. There is the question of the difference in result given by the pulley as against the car wheel, when it is considered that one runs on a fixed center and the other on a movable one, as well as being supported on its periphery. But when it is remembered that the car wheel does many times the work of the pulley the results can not be much different.

Whether it is possible in railroad service to obtain in the beginning and maintain in practice perfect conditions of this kind is a simple mechanical question. What the result would be if it were done cannot be exactly stated because it has never been tried, but it is no unknown field to work in; there is no experiment about it. If it cannot all be accomplished at once it can at least receive a share of the effort made in other and similar directions.

Ten years ago we made car wheels as many makers do to-day. Bought good material, used the best of care and conducted the whole matter as a foundry business. But seven or eight years ago we introduced methods that gave a little better information as to the quality of each wheel; the practice of finding that out in service was not exactly satisfactory.

It would take too long to follow the work up in all its stages; how one thing led to another, and so on. But some six years ago the point was reached where a definite system covered every wheel made as an individual thing. This, of course, mainly concerned the wheel as it came from the foundry. It was possible to know whether each one was of proper quality and to have full information on that matter. In the effort to improve, the mechanical features of the question were opened up and followed, until to-day the wheel leaves the foundry perfect as it can be made there. But not to go to the railroad shop to have, perhaps, fifteen minutes spent in putting a hole through it and pressing it on an axle as the sum total of preparation for the most arduous mechanical work known. It goes to the machine shop, where it is bored out, turned

absolutely true and balanced. Many say that this is unnecessary and useless work; that it decreases the life of the wheel; that the skin (?) on the chilled surface is the vitally valuable thing that must not be touched by the maker; that a difference in diameter is of no particular account; and that as to rotundity and balance, perfect castings can be turned out of the foundry. The writer has spent nearly twenty years in the foundry; learned the trade and worked at it for years; is fairly well posted in what is theory and what is practice. If the machine shop has no part in the preparation of a car-wheel for a service many times more severe than any other mechanical one, there can be little value in all the other details of mechanical work carried on there. The writer has yet to find any wheel-maker who has had a proper practical experience in the foundry say that wheels can be cast mechanically perfect. No doubt in certain instances with perfectly new tools, chills, etc., and special effort to produce the best result, castings can be made far more perfect than the average. But what about every-day practice? Chills do not always remain new; ordinary workmen must sooner or later attend to the details. Is it possible to expect a good general result from conditions that will only produce it under special treatment that cannot be given in every case?

So long as this question is determined by foundry practice alone and wheels are used as they come from the foundry, it is certain the conditions necessary will not be obtained. It is necessary to find a true center and finish a wheel from that in a mechanical manner to obtain a proper mechanical condition. One of two things will certainly have to be done. Railroads will have to adopt steel wheels, for the mechanical work on which they seem willing to spend hundreds, yes, thousands of dollars, where they question that many cents to obtain in chilled precisely what they seek in steel wheels, or, they will have to turn and balance chilled wheels. They are progressing step by step to heavier loads and higher speeds; they are forced to. They are equipping freight cars with air brakes. For what? That cars may carry more and run quicker. It is one thing to apply the brakes on a car which, with its load, weighs 30 tons, and runs at 30 miles an hour, and have a brakeman do the work of transferring the power of one man through a 1¼-inch brake-mast, with a leverage of 6 to 8 inches from center; and it is another thing on a train running at double the speed, with double the load, to put at the command of the engineer on a train of thirty cars a power of 50 000

pounds per car for instant use. Think of it! If the air brakes are in proper condition on such a train the engineer has stored up in the air tanks under the cars a total force of 750 tons available for instant use. It will probably make some difference under such conditions whether wheels are round or balanced. It is making the difference now.

The best wheel is the one that will not break, that is mechanically perfect and that will retain its original conditions for the longest time. We have proven to our own satisfaction that the chilled wheel can be made to fulfill these conditions, and have a total of 600 000 in every kind of service without one case of breakage, as proof of possible safety.

One-sixteenth of an inch chilled iron will give more wear than six times that quantity of steel found in any steel tire. It must be remembered that steel tempered and hardened into cutting tools and steel not so treated are very different things, and that the latter condition is always the one found in steel tires. Furthermore, the life of a steel wheel in the severe service of to-day is not all in the flat surface of the tire; it is largely in the flange. To provide proper flange thickness on many steel wheels, from 20 to 40 per cent. of the tire must be turned off and thrown away.

At best it is difficult to say what the entire result of the imperfect conditions referred to are in the practical operation and expenses, but as a conservative statement, based on long and careful investigation of the subject, the writer believes that with mechanical conditions such as they should be and such as can be maintained without difficulty on chilled wheels, the cost of power operating traffic carried over them can be decreased from 15 to 20 per cent. The cost of wheel service can be decreased from 25 to 50 per cent., and the saving in wear on equipment and permanent way will be in like proportions.

On the wheel subject generally a few words may be said. We have not studied and worked at it from the one standpoint of chilled wheel manufacturers, but have obtained all the information possible on the manufacture and use of the different types of wheels in all countries and on the results obtained from them. You will find in the appended table a very complete and interesting statement of results obtained from steel wheels in Germany during the past five years. It is instructive to those who consider steel wheels as the only proper thing for safety. It is right for railroads to investigate and try different articles to find out what is the best, but on this wheel question the practice is too largely

based on opinion, private and public. Railroad managers are so anxious to consult the latter that they say, as some of them have said to me, "We use steel wheels because if anything happened to one we could say we had bought the best." Why the best? Because the most expensive? That is one way to look at it; but on such an important item of expense as steel wheels it is quite important for them to know that they are practically as well as theoretically correct in their opinions.

A large consumer of steel wheels said to the writer that it took an average of two hundred per month to keep up his equipment, at a cost of $10 000 per month or $120 000 per annum, not to mention the cost of many lathes for re-turning the tires or the cost of the work; and yet he had more breakages of steel wheels than he ever had of chilled wheels in like periods. He said that if it were not for "public opinion" he would go back to chilled wheels. Well! Consulting public opinion at a cost of $120 000 per annum for a cause he personally did not believe in, could not be very satisfactory business.

There seems to be some inexorable law that ties some purchasers of chilled wheels down to a price that would not buy common iron castings, and that compels them to hold all wheels of that class as of one value and quality. Thirty-three-inch wheels, weighing 600 pounds, are sold commonly to-day for prices that do not net the maker eight dollars, freight and other expenses considered—a net price of $1\frac{1}{3}$ cents a pound. The best car wheel iron cannot be bought for such a price, and just how the purchaser expects the manufacturer to furnish fuel, merchandise of all kinds, plant, labor, and the general expenses of the business, and produce a good article, is a hard question to answer. Fortunately all wheel buyers do not follow such a practice, but so many do that the others think a dollar or two more should produce something extraordinary, when as a matter of fact they pay more per pound for their brake-shoes than they do at the highest price for wheels. If improvements in this matter are to be made, it must be considered from all standpoints and the same influences brought to bear upon it that have produced the highest type of modern engineering—the modern railroad.

The "connecting link" must be made as perfect as the parts it connects.

STATEMENT OF STEEL TIRES BROKEN ON RAILROADS OF GERMAN EMPIRE IN YEARS 1884 TO 1889, INCLUSIVE.

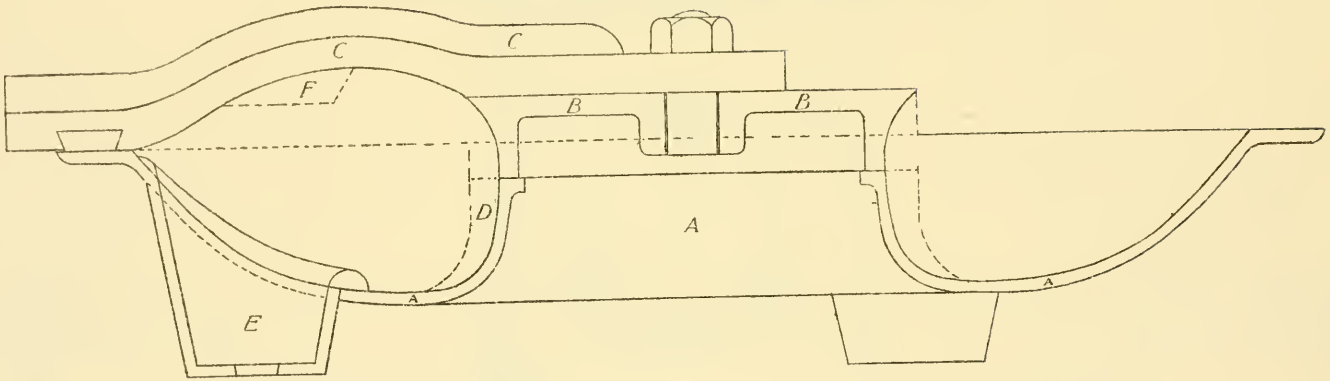
Years.	CAST STEEL.			FUSION STEEL.			MARTIN STEEL.			MANGANESE STEEL.			BESSEMER STEEL.			OTHER STEEL.			PUDDLE STEEL.			Number of Steel Tires on hand and in use.	No.	Per Cent.
	On hand and in use.	Broken.		On hand and in use.	Broken.		On hand and in use.	Broken.		On hand and in use.	Broken.		On hand and in use.	Broken.		On hand and in use.	Broken.		On hand and in use.	Broken.				
		No.	Per Cent.		No.	Per Cent.		No.	Per Cent.		No.	Per Cent.		No.	Per Cent.		No.	Per Cent.		No.	Per Cent.			
1884.....	106 838	259	0.24	197 573	298	0.15	53 922	52	0.10	1 835	...	....	380 189	834	0.22	6	...	.....	201 582	1 410	0.70	941 945	2 853	0.30
1885.....	94 365	370	0.39	221 212	464	0.21	66 848	107	0.16	1 464	6	0.41	402 140	1 208	0.30	302	16	5.30	173 351	1 378	0.79	959 682	3 549	0.37
1886.....	96 099	294	0.31	189 688	500	0.26	93 694	177	0.19	2 039	...	....	453 857	1 771	0.39	244	32	13.0	157 015	1 214	0.77	992 636	3 988	0.40
1887.....	99 768	267	0.27	129 232	304	0.24	171 370	169	0.10	1 767	17	0.96	471 030	1 399	0.30	214	28	18.08	134 615	780	0.58	1 007 996	2 964	0.29
1888.....	92 203	348	0.38	115 617	380	0.33	204 932	241	0.12	1 419	18	1.20	510 925	1 998	0.39	170	68	40.0	115 887	821	0.71	1 041 135	3 874	0.37
1889.....	64 359	174	0.27	140 635	311	0.22	284 815	241	0.08	1 251	...	....	510 985	1 909	0.37	440	65	14.8	100 456	680	0.68	1 102 941	3 380	0.31

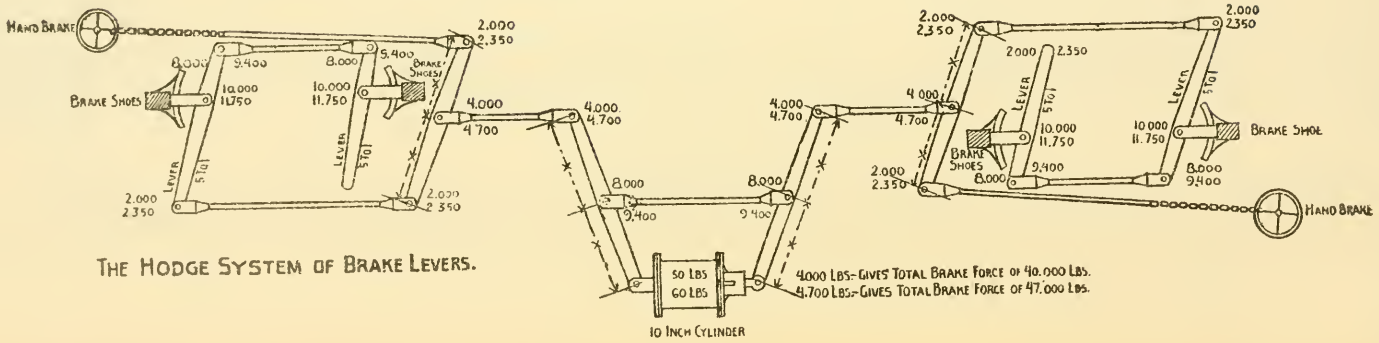
NOTE.—Plate I is a part section and a view of the 33-inch 600-pound car wheels manufactured at the New York Car Wheel Works at Buffalo.
Plate II shows the "hub" and "sweep" for the same, and also the strains in the various portions of the Hodge system of Brake Levers.
Plate III shows the arrangement of the mould for a 33-inch wheel.

33-inch 600 lb. Car Wheel.

33-inch Core Pan,

HUB AND SWEEP.





THE HODGE SYSTEM OF BRAKE LEVERS.

PLATE VII
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXV. Nº 487
GRIFFIN ON CAR WHEELS.

33-in. Chill, Cope and Nowel,

SHOWING PATTERN IN POSITION

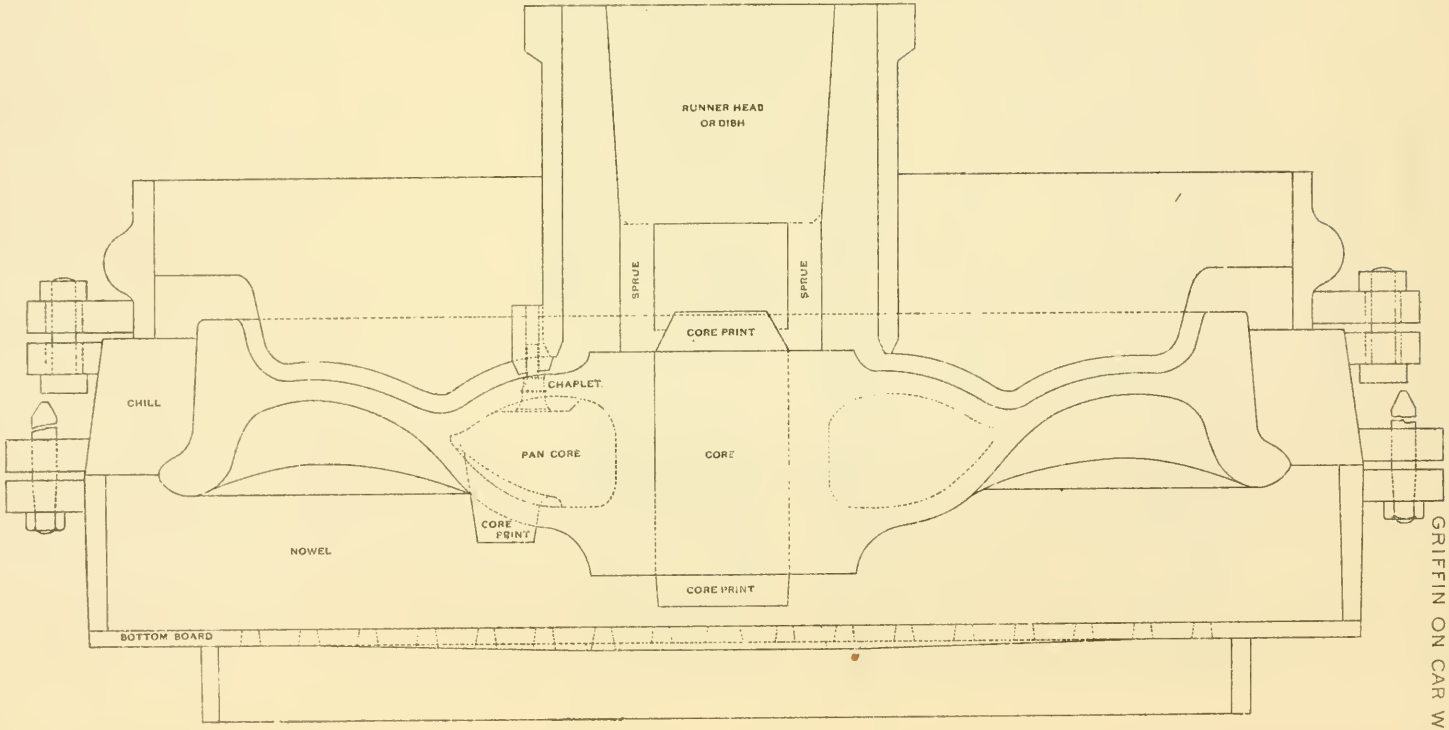


PLATE VIII
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXV. No. 487
GRIFFIN ON CAR WHEELS.

DISCUSSION.

GEORGE B. NICHOLSON, M. Am. Soc. C. E.—Mr. President, I think we have all listened to that paper with a great deal of interest. The author has given some very valuable information that I have not hitherto had access to; I shall take pleasure in studying it. There is one question I would ask—if it is in the paper I did not catch it—to what depth does the chill go in the car wheel?

Mr. GRIFFIN.—I said in the paper, the depth of chill in wheels is five-eighths of an inch.

Mr. NICHOLSON.—In turning down wheels that are brought to the shop how much of the original five-eighths inch chill can we safely afford to lose?

Mr. GRIFFIN.—You would be safe in turning a ½ inch for such a wheel.

Mr. NICHOLSON.—Half an inch. That question was suggested to me some months ago by a gentleman who had a patent process for turning down wheels in which he claimed to produce a second chill ⅛ inch in thickness. In his process of turning down a wheel he produced by friction a heat on the surface.

Mr. GRIFFIN.—The chilled surface on a car wheel is caused by the sudden contact of the hot metal with the cold chill, and the result is due to the instant combination of carbon and iron at that point. Therefore, the chill is originally produced by the application of cold. The application of heat restores the metal to its original condition and destroys the chilled surface. There is no such thing possible as producing a second chill by any process that does not re-melt the iron and produce a repetition of the original conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

488.

Vol. XXV.—July, 1891.

A MEMOIR ON WATER METERS.

By JOHN THOMSON, M. Am. Soc. C. E.

WITH DISCUSSION.

That there shall be no misconception in respect of the scope of this paper, it seems necessary to amplify the above caption by saying that it is not, nor does it pretend to be, an exhaustive technical treatise upon the subject of water meters, but is to be taken within the measure of its title, that is to say: the recital of personal knowledge and experience, with illustrations, tables, data and opinions, having direct and collateral relation to the development of the present state of the art.

The more one has to do, and the more intimate he becomes with the water meter question, the more certain is he to be astonished at the amount of capital, persistent thought, time and labor that have been expended upon this branch of hydraulic engineering; which is well exemplified in the patent tables herein submitted, showing, with approximate exactness, the number and designation of United States and British patents that have been issued for inventions in water meters. Thus it will be seen that from 1837 to the close of 1890, a period of fifty-three years, the United States Patent Office granted 678 patents in this class; while from 1824 to the close of 1889, a term of sixty-five years, Great Britain granted 231 patents, or a grand total in America and Great Britain combined of 909 patents. It is estimated that about 250 meter patents have also been taken in France, Belgium and Germany.

Among the most interesting deductions to be made from these patent tables are that about 50 per cent. of the total number of patents in the United States were issued during the last fifteen years; while about 50 per cent. of the total number of British patents were issued during the last twenty-two years.

Although it is probable that, say 10 per cent., of the grand total of patents are for the same inventions, still the figures are more probably under than over the actual, as the class comprising ships' logs and speed indicators has not been searched, not to speak of the analogous classes in pumps and engines.

NUMBER AND DESIGNATION OF UNITED STATES PATENTS, ISSUED FOR INVENTIONS IN WATER METERS, FROM 1837 TO 1890, INCLUSIVE.

YEAR.	ROTARY.	PROPORTIONAL.	PISTON.	OSCILLATING.	DIAPHRAGM.
1837.....	....	....	....	....	1
1850.....	1				
1851.....	1	....	1		
1852.....	1	....	2		
1853.....	....	....	2		
1854.....	....	....	2		
1855.....	....	....	3	1	
1856.....	....	....	1	1	5
1857.....	1	....	1	2	1
1858.....	2	....	....	3	1
1859.....	3	1	1		
1860.....	....	1	1		
1861.....	3	....	3	....	1
1862.....	2	....	3	1	1
1863.....	....	1	2	....	1
1864.....	1	....	1		
1865.....	3	....	3	1	
1866.....	....	....	4		
1867.....	4	....	8	2	1
1868.....	9	....	13	6	2
1869.....	12	1	17	9	3
1870.....	14	....	16	5	2
1871.....	14	....	28	7	5
1872.....	8	....	15	5	2
1873.....	6	....	4	2	1
1874.....	11	1	7	1	3
1875.....	7	2	5	....	5
1876.....	10	1	10	4	1
1877.....	10	....	8		
1878.....	8	2	9	2	1
1879.....	7	....	6	1	3
1880.....	6	1	3	3	
1881.....	4	1	9	2	
1882.....	5	....	10	6	
1883.....	7	....	6	1	3
1884.....	5	2	10	7	
1885.....	10	4	12	6	5
1886.....	11	18	11	5	
1887.....	11	2	11	2	1
1888.....	13	....	8	3	
1889.....	8	3	4	3	1
1890.....	11	1	2	4	
Total..	229	42	262	95	50

Grand Total.....678

NUMBER AND DESIGNATION OF BRITISH PATENTS, ISSUED FOR INVENTIONS IN WATER METERS, FROM 1824 TO 1889, INCLUSIVE.

YEAR.	OSCILLATING.	PISTON.	PROPORTIONAL.	YEAR.	OSCILLATING.	PISTON.	PROPORTIONAL.
1824....	2			1869....	.....	16	1
1835....	1			1870....	.....	4	
1840....	1			1871....	2	12	
1841....	1	1		1872....	1	1	
1842....	1	2		1873....	1	2	
1849....	1	4		1874....	.....	6	
1850....	1	1		1875....	1	6	1
1851....	2	1		1876....	1	10	
1852....	.....	8		1877....	.....	4	1
1853....	.....	3		1878....	.....	8	
1855....	1	2		1879....	2	8	1
1856....	1	2		1880....	4	4	
1857....	2	4		1881....	.....	8	
1858....	2	1		1882....	2	3	
1860....	1	4		1885....	2	3	1
1861....	.....	4	1	1886....	2	6	
1862....	1	3		1887....	1	7	
1863....	.....	1		1888....	1	7	1
1864....	1	2		1889....	3	3	1
1865....	.....	1					
1866....	.....	4					
1867....	1	7					
1868....	1	6		Total.	44	179	8

Grand Total..... 231

The first patent for a water meter was issued in the United States in 1837, for a diaphragm meter, but it was not until 1850, or after a lapse of thirteen years, that the second United States patent in this class was issued. In the following year, 1851, the third and fourth United States patents in this class were issued, both to the late Captain John Ericsson, Hon. M. Am. Soc. C. E., one for a double piston meter and the other for an inferential meter. The positive uncompromising character of Captain Ericsson is well illustrated in the expressions that appear in the latter patent, wherein the operation of the devices is predicted with such a degree of certainty as might have been applied to the law of gravitation.

Of the five United States patents issued for water meters in 1859, and only about the fortieth in sequence, one was to Mr. B. S. Church, M. Am. Soc. C. E., for a rotary meter.

The first British patent for a meter, No. 4982, of A. D. 1824, is designated a "fluid meter," but it is more properly a gas meter. It has had but little influence upon the art, yet the title of its inventor may be here recorded, which is as follows: William Pontifix, the younger, of

Shoe Lane, in the City of London, coppersmith and engineer, sends greeting:

British patent No. 8393, of A. D. 1840, to John Hanson, in a meter for "gas, water and other fluids," shows a piston-valve, contained in a cylinder, and has an exterior sleeve and chamber connected to the cylinder, in a manner practically identical with the "piezometer connection" shown by Mr. John R. Freeman, M. Am. Soc. C. E., in his paper on the "Hydraulics of Fire Streams."* Whether Mr. Freeman reinvented the device (as he probably did) or adopted it, in either case its selection is equally creditable to his skill or to his judgment.

One has not far to look for a portion of the motive which preceded the applications for these patents, as it is found in the growing belief, held by many hydraulic engineers and water-works officials, that there is but one entirely satisfactory and just method of selling water to committees or individuals, which is by measurement of quantity; that it would, in its results, be equally illogical and unsatisfactory to sell gas, bread, or medicine by *pro rata* tax as it in fact usually is when water is so disposed of. Nevertheless, it has only been for a short period that any actual record has been readily obtainable capable of showing what has actually been accomplished in the introduction of meters; and which at the same time, is in the nature of a demonstration of the commercial possibilities for the future.

The accompanying tables of meters and taps, compiled from the latest and most reliable data, show the surprising fact that the total number of water meters in public use in the United States, is but 124 798. But from a careful, conservative estimate, and considering the claims of the several meter manufacturers, it is probably safe to say that at least 175 000 water meters have actually been made and sold in this country. The discrepancy between the writer's estimate and the published returns, is undoubtedly due to the difficulty and often the impossibility of obtaining replies from water-works officials; to "wear and tear;" to private ownership, and mayhap to the asserted tendency of meter makers, like pump, engine and bridge builders, to tack on a unit now and then to their enumeration.

* Transactions, 1889: Vol. XXI, page 303. November, 1889.

TABLE SHOWING BY STATES, AND IN TOTAL FOR THE UNITED STATES, THE NUMBER OF TAPS, THE NUMBER OF WATER METERS, AND THE RELATION OF METERS TO TAPS. COMPILED FROM THE *Engineering News* PUBLISHING COMPANY'S "MANUAL OF AMERICAN WATER WORKS, 1889–90."

STATE.	Number of Taps.	Number of Water Meters.	Percentage of Meters to Taps.
Maine	19 515	320	1.63
New Hampshire	14 076	1 403	9.96
Vermont	8 638	466	5.39
Massachusetts	221 610	24 912	11.24
Rhode Island	24 315	12 899	53.00
Connecticut	49 071	959	1.95
New York	410 236	30 219	7.36
New Jersey	96 408	6 608	6.85
Pennsylvania	337 040	1 263	0.37
Delaware	12 363	11	0.09
Maryland	74 165	835	1.12
District of Columbia	23 489	14	0.06
Virginia	23 140	185	0.80
West Virginia	9 991	5	0.05
North Carolina	1 940	381	19.62
South Carolina	1 067	22	2.26
Georgia	11 391	2 646	23.20
Florida	2 970	514	17.30
Alabama	7 306	1 000	13.70
Mississippi	1 250	9	00.72
Louisiana	4 660	20	0.42
Tennessee	15 748	432	2.76
Kentucky	23 361	1 311	5.61
Ohio	101 708	4 789	4.70
Indiana	23 623	822	3.48
Michigan	63 185	1 042	1.65
Illinois	154 966	4 897	3.16
Wisconsin	21 786	470	2.15
Iowa	19 281	1 691	8.25
Minnesota	12 114	466	3.82
Kansas	17 160	768	4.47
Nebraska	10 707	786	7.34
South Dakota	2 120	58	2.73
North Dakota	1 423	219	15.40
Wyoming	1 028	152	14.75
Montana	1 454	26	1.79
Missouri	57 182	4 056	7.10
Arkansas	3 538	131	3.70
Texas	19 675	1 303	6.62
Colorado	15 191	226	1.49
New Mexico	1 208	17	1.41
Washington	5 828	106	1.82
Oregon	8 745	106	1.21
California	77 954	16 103	20.68
Arizona	1 145	108	9 43
Nevada	4 425	20	0.45
Utah	3 393	1	0.03
Idaho	520	1	0.20
Total Number and Relation in United States	2 023 109	124 798	6.168

TABLE SHOWING THE NUMBER OF TAPS, THE NUMBER OF WATER METERS, AND THE RELATION OF METERS TO TAPS IN THIRTY-ONE OF THE PRINCIPAL CITIES OF THE UNITED STATES. COMPILED FROM THE *Engineering News* PUBLISHING COMPANY'S "MANUAL OF AMERICAN WATER WORKS, 1889–90."

CITY,	Number of Taps.	Number of Water Meters.	Percentage of Meters to Taps.
Albany	14 682	52	0.35
Augusta (Me.)	800	0	0.00
Atlanta	2 466	2 286	95.30
Boston	74 554	3 520	4.72
Buffalo	32 650	115	0.35
Baltimore	68 000	791	1.16
Brooklyn	82 217	2 137	2.60
Chicago	136 267	3 278	2.40
Cincinnati	33 082	1 304	3.94
Cleveland	27 516	1 644	5.97
Charleston	1 000	20	2.00
Denver	5 000	20	0.40
Detroit	31 821	50	0.10
Fall River	4 412	3 138	71.10
Hartford	5 984	345	5.85
Harrisburg	8 200	200	2.45
Jersey City	20 456	240	1.18
Kansas City	10 436	1 000	9.58
Louisville	11 001	678	6.17
New York	180 000	18 211	10.12
New Orleans	4 178	18	0.44
Nashville	4 496	15	0.34
Newark	19 854	501	2.54
Pittsburgh	17 681	45	0.25
Philadelphia	172 604	267	0.15
Providence	13 643	8 094	59.30
Richmond	9 606	85	0.88
Rochester	21 000	1 800	8.59
Springfield	5 147	556	10 90
San Francisco	30 000	12 750	42.50
Washington	23 489	14	0.06
	1 072 242	63 174	5.885

Now, divide the number of meters by the years of patent monopoly, say, forty, and we will find an average yearly sale of about 4 400, although it is not overlooked that from 80 000 to 100 000 of these meters have actually been sold during the past ten years, or an average of from 8 000 to 10 000 meters a year during that period. This fact alone would indicate a very rapid growth in sentiment favorable to the introduction of meters. Yet, taken from even its most favorable point of view, this is certainly not in itself such a showing as would be likely to so impetuously lead on the usually, largely expectant inventor.

Let us, however, refer to the other columns of the table and we will see that the total number of taps, or separate house services in the United

States, is 2 023 109; that, therefore, the relation of meters to services is nearly as but 6.2 meters to 100 taps.

In the table of thirty-one cities, the unexpected fact is brought out that the relation of meters to taps is less than that of the country at large, a result exactly contrary to what was anticipated.

After due consideration of the foregoing, it will probably at once be apprehended wherein lies the commercial incentive to the engineer to produce a meter of which he probably predicates in advance—in the glow of his youthful enthusiasm and self-confidence—that it shall be capable of successfully meeting the most exacting criticism; shall even be better than the best, and consequently will be certain of an introduction and sale equal to that residual of unserved taps which even yet amounts to the very substantial sum of 1 898 311 of the meter services of the United States.

The subdivision of the class of water meters, as it appears in the records of the Patent Office, is of but little value in an engineering sense, and must often be as misleading and troublesome to the office as to its clients. In the broadest sense there are but two types of water meters, those embodying the positive displacement principle and those embodying the inferential. By positive displacement is meant the successive filling of a chamber and the displacing of the fluid therefrom by mechanism capable of being acted upon by pressure, as in the instance of a cylinder and piston. By the inferential type is meant any device, as a current wheel, turbine, worm, or gauge, capable of being operated by, or in any manner responding to, the dynamic action of fluid; and which thus infers the total quantity, not by measurement of bulk but by lineal velocity or by pressure. Each of the foregoing types is properly subject to many subdivisions, and such a table might be here submitted but for the fact that established usage would nullify any value it might possess.

In respect of the drawings herewith incorporated, acknowledgment is only due to a collection of "meter literature," the gathering of which has extended over several years, consisting of trade-catalogues, detached plates of "proceedings," United States Patents, "Reuleaux's Kinematics," etc., of which free use was made both by direct transposal and by tracing. The several illustrations have been most carefully selected; the aim being to make a fair representation both of the growth and of the present state of the art, but without undue amplification of space and detail.

Thus in Fig. 1 the novelty consisted in the spring, *e*, acting upon the blade, *d*, ensuring end contact against the perimeter of the cylinder. It is representative of a class of movements that have been variously modified, upon which many United States and foreign patents have been issued, but has never reached any commercial importance, except to absorb considerable capital in useless experimenting. In fact this same device, but in better and more practicable form, was patented prior to this by Woodcock, in England, about 1840, while within the last decade it again came to the fore, hailing this time from the State of Pennsylvania, and it is said that about $50 000 was expended to ascertain that, although an old descendant, it was not a survival of the fittest. The device has the serious defect of combining small displacement capacity with large area of obstruction, that is the revolving block, *C*, which is mounted upon pivots. Hence, if the apparatus be reduced to compact dimensions, a high rate of discharge soon destroys it, in that the entire sum of the pressure, caused by the obstruction, is transmitted to and is borne by the pivots.

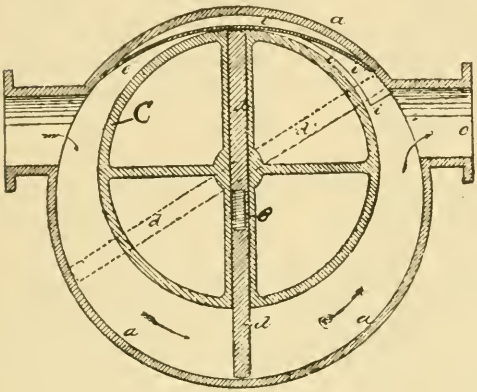


Fig. 1.

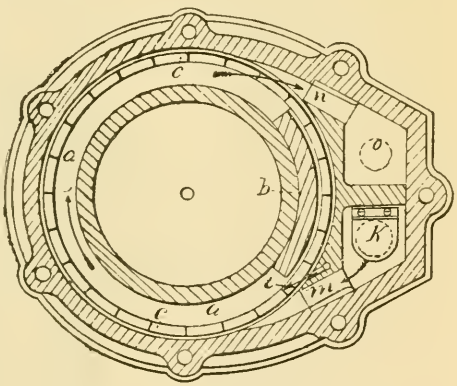


Fig. 2.

In Fig. 2, two novel features are presented: First, the loaded flap valve, *K*, in the main inlet chamber, causes a jet, *i*, of high velocity to impinge against the vanes, *c*, to start the "paddle wheel" at low rates of discharge. Second, the waterway, which follows around the semi-

Fig. 1.—Positive Rotary Meter. U. S. Patent of W. Sewell, February 5, 1850.
Fig. 2.—Inferential Rotary Meter. U. S. Patent of Capt. John Ericsson, Dec. 9, 1851.

circular chamber, *a*, entering and leaving it tangentially on the same plane. Many modifications of this device have been patented, but it has had little or no practical application in the United States.

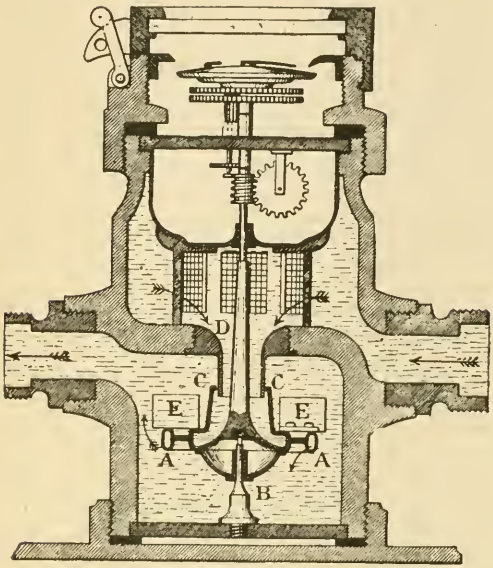


Fig. 3.

In Fig. 3 is shown a detail sectional view of Siemens' inferential reaction meter. This was an adaptation from the old Barker Mill, or "Scotch Turbine." The object of the wings, *E*, is to act as a brake, causing increased resistance and decreased speed of revolution at high rates of flow. Fig. 4 also shows a Siemens' inferential meter, but of the direct impact, or "paddle wheel" type.

Tylor's inferential meter, Figs. 5 and 6, Plate IX, of the direct impact type, presents several modifications over that of the Siemens', notably in the method of discharging the water radially from the vanes and in the same plane, as in Ericsson's. The screw, *F*, is for the purpose of increasing or decreasing the speed of the fan by controlling the size of the jet; which acts in a direction opposite to the revolution of the paddles

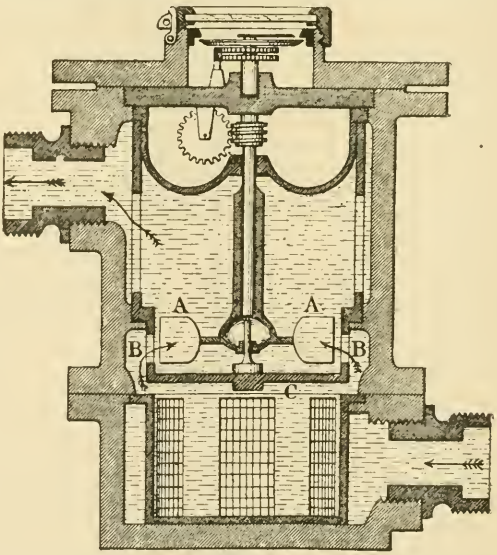


Fig. 4.

Fig. 3.—Siemens' Reaction Meter.
Fig. 4.—Siemens' Direct Impact Meter. Vertical center sections of the Reactive and Impact types. Manufactured and very extensively used in Great Britain and Europe.

B, and by which means the register may be properly rated. A recent improvement in Tylor's meter consists, as shown in the detached details, in twisting the blades of the paddle-wheel or fan in opposite directions; thus forming the wheel from one part, and overcoming the tendency to thrust upward or downward, which would otherwise exist if twisted in one direction only. It is common practice to fill the reservoir, *O*, with oil, for the purpose of lubricating the pivot and also to increase the buoyancy of the wheel. Many thousands of these styles of inferential meters are in use abroad; the manufacture of which has been developed to a high degree of excellence.

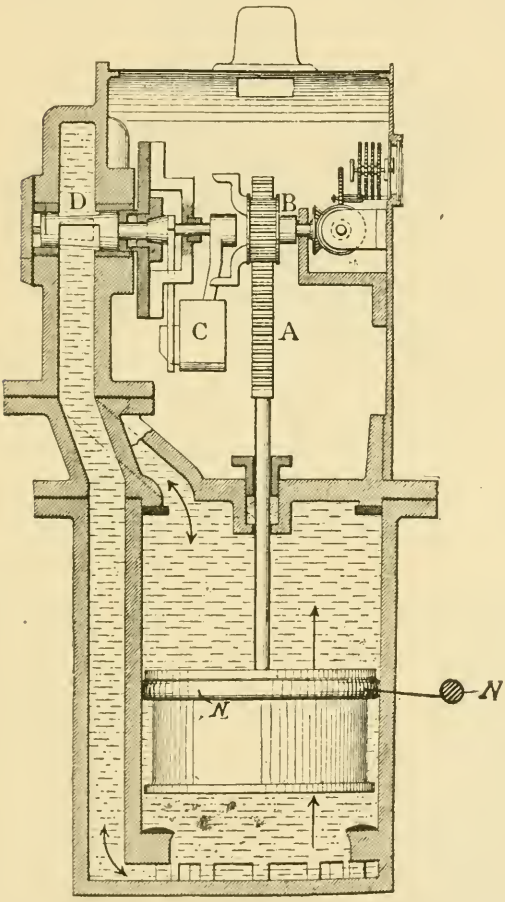


Fig. 7.

In Figs. 7 and 8 are presented two representative meters of the positive single piston type, but differing widely in detail. In the Kennedy, the piston is nearly as long as its stroke, packed by means of a ring, *N*, of pure gum rubber, which rolls between the piston and the cylinder, insuring an absolutely tight joint with but little friction. The register and valve mechanism is contained in a separate chamber, *A*, from which water is excluded. The plug valve, *D*, is operated by the vibrating weight, *C*, the weight being primarily actuated by the piston rod. The extent

Fig. 7.—Vertical center section of Kennedy's single piston meter. Valve operated by a vibrating weight. Register mechanism records the total travel of the piston. Made in Scotland and Europe.

of the travel of the piston (not the number of its reciprocations) is transmitted to the register by the rack and pinion, *B*, and a system of ratchets and pawls. Hence, any change in the relation of the speed between the action of the weight by gravity and the movement of the piston due to various rates of delivery, will not materially affect the registration. This meter has never been introduced to any extent in this country, but is largely used abroad; and although it is possessed of remarkably interesting features and is a very accurate meter, its accuracy seems to have been obtained at the expense of every other desirable element, as it is bulky, noisy and expensive. In the Frost meter, the main piston is packed, like a ram, with cup-leathers, *I*. The auxiliary piston, *C*, is also packed with leather; its function being to operate the main slide-valve, *D*. The auxiliary piston is controlled by the primary slide-valve, *A*, operated by the piston-rod, *B*. The valve and register mechanism are contained in the inlet water chamber, *P*. The register, by means of the pawl and ratchet, records each alternate stroke of the main piston; hence, unlike the Kennedy, if the strokes vary, for any cause, the registration will also be at fault. This meter, under several detail modifications, has been quite largely manufactured in Great Britain and France and to a limited extent in the United States.

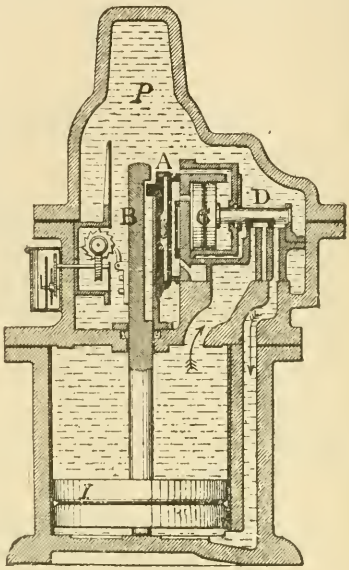


Fig. 8.

Fig. 9, Plate X, represents a meter exhibited at the Paris Exposition of 1889, manufactured in England. The writer is not aware to what extent it has been introduced abroad. Its manufacture was undertaken, with some detail modifications, by an American Company about two years ago, but has recently been abandoned. The peculiarity of this device consists in the vanes, *V*, which are caused to vibrate on their axis by means of the cam, *C*. Hence, the operation is produced by the difference of pressure between the flat and edge surfaces of the vanes, as V^1,

Fig. 8.—Vertical section of Frost's positive single piston meter. Valve operated positively by an auxiliary piston of relatively small size. Register records the number of piston strokes. Made in England, Europe and United States.

V^3. It appears that the same objection found against Fig. 1 would be equally applicable to this.

Figs. 10 and 11, Plate X, represent analogous devices whose generic base may be readily discerned in Fig. 1, from which the sliding and rotating wing has been expanded to the sliding and rotating piston, *Z*. But in consequence of the greater displacement capacity and the larger area of bearing surface of this device, it is not subject to the criticism made against Figs. 1 and 9. It has been quite extensively introduced in England and to a limited extent in the United States.

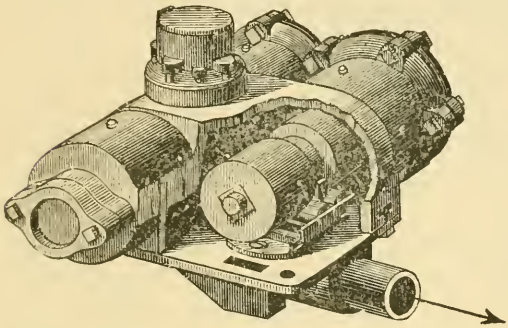


Fig. 12.

In Fig. 12 is presented the original type of duplex piston meter, which to all practical purposes was in fact the pioneer meter of the United States. It is still manufactured in essentially its original design and has otherwise been copied and modified. Were this instrument provided with registering mechanism embodying the principle of Kennedy's—it now records the number of piston strokes—it would present about all the most desirable features that could be tabulated in favor of this type of meter.

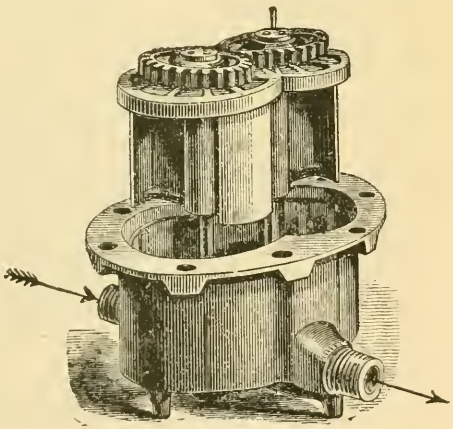


Fig. 13.

In Fig. 13 is shown a representative positive rotary meter, quite extensively manufactured in this country. And while it is open to the criticism quoted against Fig. 1, it is yet due to truth to say that careful workmanship and skillfully selected material have combined to largely overcome in practice the theoretical objections to its

Fig. 12.—Perspective exterior and interior view of Worthington's positive duplex piston meter. Made in the United States.

Fig. 13.—Perspective detail view of positive rotary meter. Made in the United States, and known as the "Union Rotary."

defective principle; as the meter in many instances has proved accurate and durable. Its origin is easily traced to the device shown in Fig. 19.

Figs. 14 and 15, Plate XI, show a positive single-piston meter similar to that of Fig. 8, except in its valve-mechanism. In this device the two pairs, *K*, of coupled puppet-valves were actuated by the spring-toggle, *J*, and vibrating lever, *L*. From $30 000 to $40 000 was expended upon the development of this meter before it was ascertained that it was impracticable and would have to be abandoned. There were, and there demonstratively are, a number of good and sufficient reasons therefor; but the object of this illustration is to explain one of the principal causes of its failure whereby to show how easy it is, even for meter experts occasionally, to primarily fail to observe what afterward appears to have been constantly obvious, too ridiculously simple to have escaped detection. The reciprocations of the slotted piston-rod, *E*, acted upon the lever, *L*, to cause its vibration. Now, it is to be noted that the tension of the spring-toggle is upward, acting through the lever directly upon the boss, *P*, to force the valves to a tight seat. When the piston-rod engages the lever, however, the tension of the spring-toggle would be transferred from the valves until the knife-edge bearing, *r*, was carried past its center, when the spring would act with great rapidity to reverse the valves. During the time of the described action, the valves were relieved from the pressure of the spring and were hopefully expected to be retained in proper postion by water pressure, more or less variable, acting upon the seated valves; but when a series of conditions, or "happens," so to speak, combined to work in unison, the uncontrolled valves would be unseated, and the result would either be vibrations and water rams of great rapidity and intensity, or a total stoppage of the meter.

The meter illustrated in Figs. 16 and 17, Plate XI, brings prominently to the mind of the writer the remembrance of an amount of labor, worry, vexation and disappointment which had but one compensation, that common to us all, he apprehends, the education due to a flat, uncompromising failure. And although he has never since been able to look upon these drawings without a feeling of regretful self-reproach at the utter depravity therein contained, and the inane stupidity of its author for not having more promptly discovered the default, he has yet concluded to here refer to it for the possible good of those whom it may concern. It may be said that this machine, under favorable conditions, was almost theoretically perfect in action and measurement. The diaphragm,

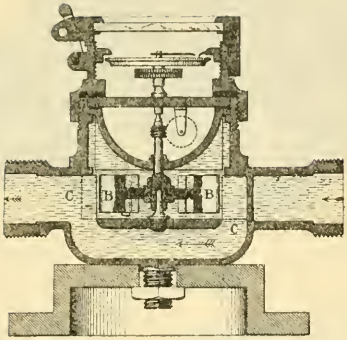
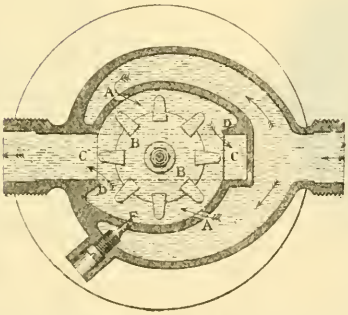


Fig. 5.

Fig. 6.



Figs. 5 and 6.

Vertical centre and horizontal sections of Tylor's Inferential Rotary Meter, Impact type. Made in England.

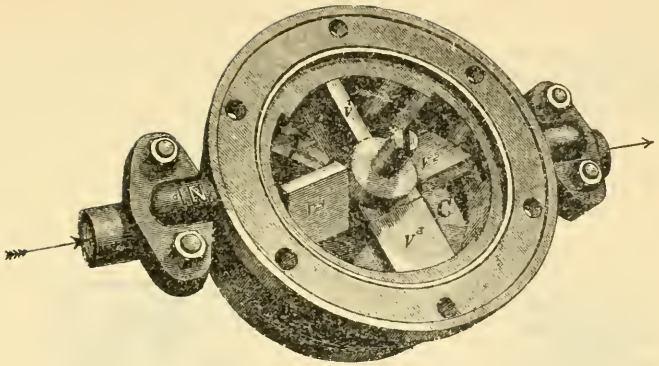


Fig. 9.

Positive Rotary Meter with Vanes vibrating axially. Made in England where it is designated the "Differential;" has also been made in the U.S. as "Niagara".

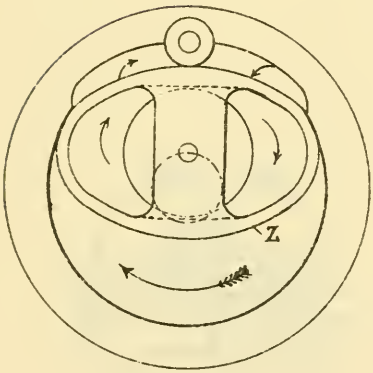


Fig. 10.

Positive Rotary Meter with rotating and sliding piston. Made in the U.S. and known as the "Undine" Patented by E. C. Terry, April 11, 1882.

Fig. 11.

Positive Rotary Meter with sliding and rotating piston. Made in England where it is known as "Kent's Uniform".

C, was of gum-rubber, usually about $\frac{1}{16}$ inch thick, but the mechanical resistance and balance of internal pressures were so nicely adjusted that a $\frac{5}{8}$ inch meter has been experimentally operated under 30 pounds pressure, at a flow equal to 1 cubic foot a minute, with an elastic gum-rubber diaphragm no thicker than a sheet of ordinary writing paper. It was decided to employ a system of piston valves because these could be the most perfectly balanced to the impact and reactive effects of the dynamic condition. The piston-rod, *h*, acting upon the lever, *l*, reversed a pair of coupled valves, *m*, which controlled the supply to an auxiliary piston, *n*, it in turn acting to reverse the main valve, *o*. The conditions in the water ways, necessary to the employment of the piston-valves, were exceedingly difficult to devise; and even when these conditions had been discovered it was found that the casing could not be cast with cores, which resulted in the expedient of constructing it in separate horizontal sections and afterward soldering them together, as at *t*, to form a single part. This mode of forming the casing proved to be entirely satisfactory, and less expensive than ordinary cored castings. The object of forming the main valve, *o*, with the zigzag edge, as shown, was to gradually close and open the ports to permit rapid reversal without excessive pulsation in the pipe. The auxiliary piston also acted in a similar manner to that of the main valve, and metallic impact was avoided by means of water cushions. In these particulars the device was in fact successful, and several hundreds of these meters, under favorable conditions, performed satisfactorily; but it was found that when, say, six meters would register and otherwise perform as desired, the seventh would be troublesome or entirely fail; it would be as if possessed of an evil spirit. Inspection of the mechanism in operation through heavy plate glass, under actual working conditions, revealed nothing; the vibrations of the valves either appeared as perfectly rhythmical as the oscillations of a pendulum, or if they failed there was no evidence as to the cause. Finally, after weeks and months, a theory was evolved; the experiment to prove the theorem came readily enough; the mystery was solved and chagrin came with it; for it was immediately evident that, under the conditions required, the fault was without remedy. And this is what the author had learned: That between the inlet compartment, *v*, and the outlet compartment, *w*, at rapid rates of discharge, there was sufficient difference of pressure to cause a current of so high velocity to pass the closely but necessarily freely fitted valves, as to

carry the valves in the same direction as the current. That was all; but it was enough.

In Fig. 18 we have an action first brought into prominence by Ericsson's application of it to marine engines. Its next important use, in popular consideration at least, was in the celebrated "Navarro" meter, of which about ten thousand of the 1-inch size were made for New York City, and for which the city ultimately paid over $1 200 000. The device is obviously inferior to a cylindrical piston and cylinder, and has properly had but little attention from mechanicians.

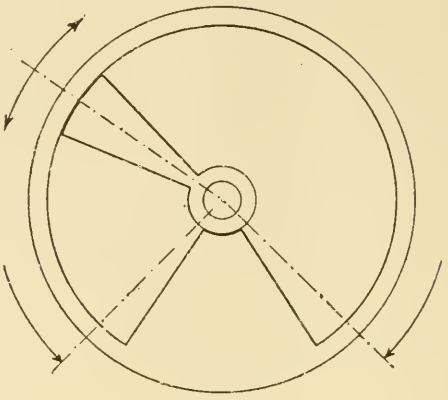


Fig. 18.

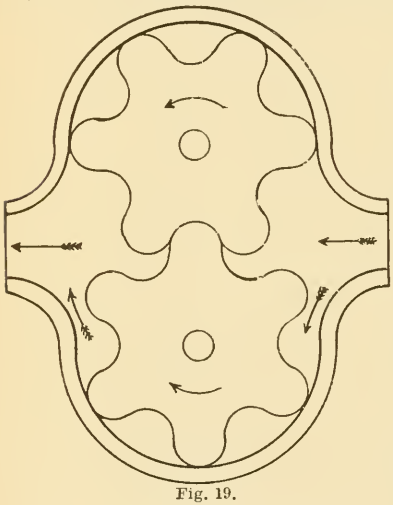


Fig. 19.

Fig. 19 represents the origin of such meter actions as those analogous to Fig. 13, the "Shedd," and many others. This may be regarded as the genus of this type, for it has probably been more closely studied, and has been the subject of more modifications than any other rotative movement, both in its application to pumps and engines as well as to meters.

Fig. 20 represents a device which was of no practical importance in the art until modified and adapted for use as a water meter by Nash, Gaskell, Tilden and others. This diagram shows pistons, *c*, of two, three and four wings or lobes working in a casing, *d*, having three, four and five spaces. The displacement is positive.

In Fig. 21 it is to be observed that there are here shown seven spaces, *d*, and six "teeth" or lobes, *e*. When a tooth is forced into a space the

Fig. 18.—Diagram of positive vibrating or wing piston movement. A very old device used in engines, pumps and meters.

Fig. 19.—Diagram of positive rotary action. A very old device, whose generic base is found in any pair of spur gears. It is known as "Pappenheim's" Wheels.

fluid will be displaced therefrom. The piston, *c*, contained in the casing, *r*, partakes of two movements, one of rotation, *t*, the other axial revolution of the spindle, *i*. The axial motion is differential, derived from the rotary; the relation, therefore, being as the spaces to the teeth, hence each tooth will eventually enter all the spaces. The movement is identical with that of a pair of external and internal gears, otherwise termed an annular train.

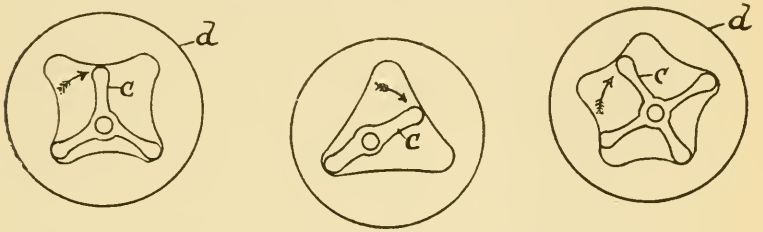


Fig. 20.

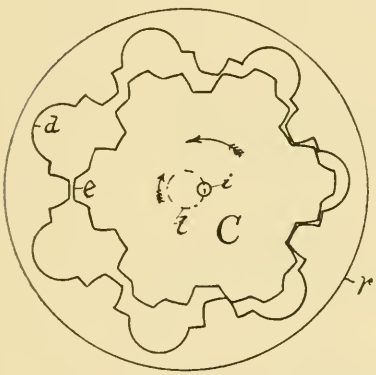


Fig. 21.

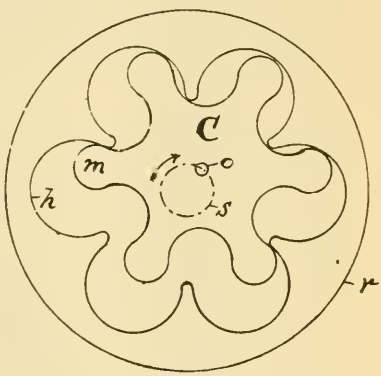


Fig. 22.

Again, in Fig. 22, it is to be noted that the number of spaces, *h*, and teeth or lobes, *m*, are here equal to each other. Each tooth or lobe "rotates" within a space, receiving and displacing therefrom. There is no axial revolution, but the spindle, *o*, describes a circle, *s*, due to the translatory movement of the piston, *C*; hence each tooth continually operates within the same space. Both of the devices shown in Figs. 21 and 22 have proved very satisfactory in American practice, and many thousands of meters employing these devices are now in use. It is to

Fig. 20.—Modified diagrams of Galloway's positive rotary action. Patented in Great Britain, 1846, as a steam engine. The generic base of this device is found in any pair of internal and external gears.

Fig. 21.—Diagram of positive rotary meter, devised and patented by Lewis H. Nash. Derived from the movement shown in Fig. 20. Made in the United States, and known as the "Crown."

Fig. 22.—Diagram of positive rotary meter, devised and patented by James H. Tilden. It is also derived from the movement shown in Fig. 20. Made in the United States, and known as the "Hersey."

be said that the spindles, in neither of these devices, have any important function other than to transmit motion to the register, the thrust of the pistons being borne by the exterior surfaces of the teeth. At the same time they are open to a like criticism urged against Fig. 1: that is to say, small displacement with great area of obstruction; but with this notable exception, undoubtedly the key to their success, namely, that the area of their bearing surface is equal to that of the obstruction.

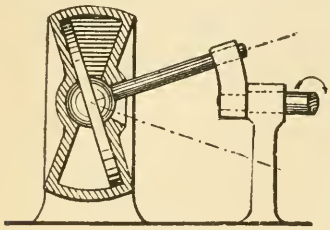


Fig. 23.

In Fig. 23 is presented the original of the now fairly well-known "disk-action." This is really the oscillating movement of a disk controlled by a ball-and-socket joint, although it is popularly designated as a rotary motion. The disk-action was first taken up in the United States about ten years ago by the Colt's Patent Fire-Arms Manufacturing Company, of Hartford, Conn., whose Mechanical Engineer at that time was Professor Charles B. Richards, until recently a member of this Society. In the instance of the Colt's Company, the device was employed in a steam engine, the function of the disk being to transmit the thrust of a series of pistons, to convert reciprocating into rotative motion. The writer's relation to the Colt's Company kept him, so to speak, in touch with the development of this engine, and the very able work expended thereon by Professor Richards, all of which resulted in

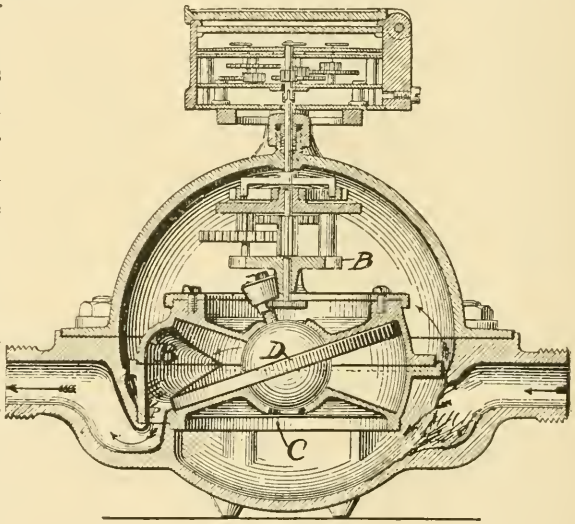


Fig. 24.

Fig. 23.—Diagram of positive oscillating movement, usually termed the "disk action." Patented in Great Britain, first in 1830, by the Dakeynes, Davis, Bishop and Ericsson, as a pump and engine.

Fig. 24.—Vertical center section of positive oscillating or disk meter. Made in the United States, and known as the "Thomson."

my being smitten with the "meter fever" and also with the controlling advantages believed to inhere in the disk-action for such a purpose. In fine, the subsequent result was the meter commercially known by my name, and here illustrated in Fig. 24. The improvements made in this application of the device referred to the internal casing, *C*, form of ports, *B*, method of controlling the movement of the disk, *D*, arrangement of gearing, *P*, etc. Unlike other actions, the displacement capacity is much greater than the obstruction. A meter employing the disk-action was made in England several years ago, but its manufacture was abandoned. The structural difficulties of this device are very great, and have probably been the chief reason why it has not received more extended attention. Several thousands of meters, made to the design here shown, have given excellent satisfaction in practice, proving remarkably accurate and durable. They are very compact and light, have a high capacity, and are noiseless in operation. At the Paris Exposition of 1889, a 1-inch meter of this design was shown in operation under a head of less than 1 inch.

To successfully work out what has well been termed the "meter problem" requires most careful consideration of and adaptation to several important conditions. On the one hand the practical and technical requirements of water-works officials, on the other hand the engineering and commercial necessities of the manufacturer. Though distinct, all are interdependent, and failure to meet one usually means failure in the whole. It is now proposed to take up very briefly these phases of the subject.

In Europe the necessity for economy in the public water service is probably accountable for the much greater use of meters which there exists than in America. It is certain, too, that the employment of meters abroad has resulted in a satisfactory reduction of the water-waste, notwithstanding the fact that this has been largely accomplished by the use of the low-priced and comparatively inaccurate types of inferential meters. Hence, there is here presented the apparent necessity for close measurement, but with satisfactorily economical results being derived from the employment of what is generally conceded to be the most inaccurate type of domestic water-measuring instruments.

With us, however, it is but a slight stretch of poetical license to say that, when the water-works engineer, who for years has stood complacently by while water has been wastefully served at from 75 to 150 gal-

lons per capita per day, who then suddenly concludes to apply a few meters to check this lavishness, his economy goes not only to the dollars but to the pennies; the meter is required to measure down to drops, and a standard of accuracy is not infrequently set up which but few laboratories have the means of verifying, and fewer water-works employees the ability to carry out.

The point of this is that for all practical purposes, in ordinary public service, a meter which would register to within 5 to 7 per cent. of accuracy, between fair minimum and maximum rates of discharge, is, in the writer's judgment, as in that of many others, amply accurate to effect the desired purpose. And when our water-works officials will have arrived at the same conclusion, meters may then be purchased at a discount from present net prices of from 20 to 25 per cent. Furthermore, such a standard of accuracy would, in ordinary practice, result in decreased cost of maintenance and increased life to the meter, because of the practical conditions of service under which meters are frequently set.

These conditions, to the novice, appear somewhat severe; for it is usually expected of a meter that it will operate with equal uniformity, durability, and accuracy, whether the water is from a crystal spring, from the bottom of a settling basin, or from a long dormant "dead end" filled with gravel, scale, paint, and decayed vegetable matter. It must be capable of standing idle for six months, while the proprietor is absent on a European trip, and start with promptness and accuracy, to full capacity, upon the instant of his return; it must not obstruct the flow; it must not grind or pulsate, to the annoyance of fine-strung nerves; it must be able to pass and measure the agile eel or the more phlegmatic infant alligator; for the owner it must record full measure, but to meet favor with the householder it should always register rather under than over the actual quantity. In fine, like the great apostle, it should be all things to all men.

One of the perplexing difficulties to be met with in meter practice is the determination of the maximum rate of discharge for the several sizes of meters, as this involves not only the limit of safety but of durability, and it is clear that upon this may devolve the commercial success of the undertaking. In casting around for a basis of computation one might naturally turn to what is usually regarded as good engineering practice in respect of water-pipes, but when a superintendent fits up his bench-testing appliances under such conditions as to blow through a meter the

greatest possible quantity in the shortest possible time, the meter man finds himself in this dilemma: either he must design his meter to stand excessive duty, or he probably will not long be a maker of meters.

One of the most difficult experiences met with in practice has been to make clear to many water-works superintendents that, in selecting the size of a meter, conditions of operation have frequently much more to do with the quantity discharged than the mere matter of pressure. As bearing upon this subject, the author submits the following paragraph which he put into a trade catalogue about two years ago:

Regarding the matter of "pressure" as affecting the volume delivered, we have found a surprising extent of misapprehension, or looseness in the use of terms, in this respect, as, for instance, when it is advised that "Piston meters of rapid motion should not be used under pressure exceeding 50 pounds," etc. Such expressions have been well termed "using words without fully rendering them into thoughts," for it would seem that the merest tyro in hydraulics ought to appreciate the palpable fact that a meter may be used more destructively under certain conditions, at a head equal to, say, 15 pounds pressure to the square inch than under other conditions at 50 or 150 pounds pressure. The whole matter may be briefly explained under two simple conditions, namely: First, the greatest static pressure should be known, to the end that suitable provision therefor may be made by the manufacturer in the cylinders, flange bolts and gaskets; and, second, the greatest possible volume, or quantity, that will ever be required to pass the pipe in a given time, where it is intended to place the meter, should also be known, that the proper size may be selected. It is thus an equation of quantity into time, and these are determined by the simple element, pressure, and the complex but all-important factor, conditions of operation. The cases are most frequent in which a meter smaller than the pipe may be properly set, while in other instances—as free outlet, no "back-pressure," under-suction, large supply, or in circuit with a motor—a meter larger than the pipe may properly be required. We repeat, it is the volume of water delivered per minute or hour of time which should determine the proper size of meter to be selected, as upon this will depend the number of motions its mechanism must make in a given time. A water meter is like a steam-engine without a governor; it is but fair to expect, therefore, that they shall be set under such conditions as to preclude a rate of operation which, in the engine would be dangerous, and in the meter would be destructive of itself.

From whence it follows that probably three-fourths of the meters in public use to-day are considerably underworked; and that, if proper judgment could be depended upon in selecting the capacity of the meter

to the duty to be performed, still more compact and less expensive nominal sizes might be the result.

Although the foregoing can hardly be expected to be taken as a compliment to the efficiency of water-works employees in general, and motives of policy alone would determine omission thereof, it must yet stand, in that, first, it is well within the truth, and, second, because it is intimately related with the future of this subject; for in the writer's belief and judgment, the proximate increase of efficiency in water-meter practice will come quite as much from the better knowledge and practice of water-works employees as from direct improvements in meters by manufacturers and engineers.

But these strictures against water-works officials and employees are not to stand alone; for the author believes that more legitimate failures in meters can be directly traced to inattention in the working up of details by meter manufacturers, than to their failure to select sound principles upon which to build. It is no salve to the vexation of a traveler who has missed a train, because his watch has stopped, to inform him that the watch failed of its duty simply because of interference between the hands. It indeed might have been more satisfactory for him to have learned that the main-spring or balance-wheel pivots were broken. And precisely so in a meter. The failure that comes from a faulty gear train; a defective screw, or an unskillfully packed stuffing box, is quite as likely to militate against the system as if the controlling principle itself were wrong. It is as well authenticated as any of the grand elementary principles, and may stand the present digression, that it is the obscure details, the little things, because unrecognized or passed by as "good enough" or as of no consequence, which most often stand as the dividing line between that which shall be successful and enduring, or unsuccessful and short-lived.

As to whether there has been, during, say, the past twenty to twenty-five years, any marked increase of accuracy in water meters, considered simply and solely on this feature, the author would answer, No. As an instance, reference may be had to the Kennedy meter, described in the Proceedings of the British Institute of Mechanical Engineers, so far back as 1856, and which yet retains its old time prestige of being one of the most accurate of water-measuring instruments. But what has been accomplished by modern workers, and to a marked degree, has been to retain a high standard of accuracy, between wide ranges of discharge,

combined with reduction of weight, bulk, first cost and cost of maintenance.

Were the question asked, Has any generic invention of marked importance been made in water meters during, say, the past twenty to twenty-five years, the reply would promptly be made, No, there has not; for it is a well known fact that the water meters now in the market, both in the United States or in Europe, have either survived the period named or are modern adaptations of, and improvements upon, old and comparatively well-known devices.

It is to a most pronounced degree remarkable with what thoroughness and inclusiveness, both in detail and scope, the mechanicians of the early part of this century took up the development of devices pertinent to meters, pumps and engines. This is a matter, so far as the author is aware, that has not heretofore been specially commented upon, and it would be an interesting comparative query to answer if any similar condition exists in other branches of kinematics. One cannot rise from an investigation of this character without a feeling of increased admiration and respect for the engineering ability of the period referred to, whose bequests often form the foundation, and are even as the retaining walls, of many modern business successes. Thus, when Siemens took the Barker Mill, a reaction motor of but little efficiency, and made of it a meter of a considerable degree of accuracy; and whose economic results in the saving of water can hardly be over-estimated, it was not a simple transfer in name but involved close and clear mechanical reasoning. In the positive piston meter of Kennedy, the rolling piston packing, and the method of registering the extent of piston strokes, are fundamentally as perfect to-day as they were over thirty-five years ago. The masterly conception of Worthington, who took a pair of simple pistons and a pair of equally simple valves, and so disposed them that one caused the operation of the other, creating the duplex type, dispensing with springs and weights, has never in these features been susceptible of improvement.

Now, while it is not to be overlooked that the production of the compact, high-speed, rotary meters of the present day would have been impossible at the time when the links were being forged to form the chain of this branch of kinematics, for the one reason, if no other, that the necessary material was not at their hands; this but quotes an additional obligation, to another art distinct and separate herefrom, to which the

modern engineer is in honor bound to make acknowledgment, that is to Goodyear's discovery of the vulcanization of rubber. Without hard-rubber, or "ebonite," the meters of Nash, Tilden, the writer and others would have been practically impossible of accomplishment; for no other combination of material, yet discovered, will in a meter so successfully withstand long severe service as rubber and the bronze compositions.

If of good quality for this purpose, hard rubber is of but little greater specific gravity than water, therefore presenting the minimum of inertia to be overcome; water is to it a fairly good lubricant; it is readily molded to the approximate, and even to the exact form desired, while it is also susceptible of being readily worked by ordinary machine processes; it is possessed of considerable rigidity and tensile strength; it is furthermore highly resilient, and probably to this characteristic is due, more than to all others, its remarkable efficiency in this art, and but for the fact that it is susceptible to the effect of hot water, becoming plastic therein, it would be in every particular, for this purpose, without flaw.

Notwithstanding what has been said, and while it may illy behoove one to claim originality of production for that which has been a free gift of time, modern workers need not hesitate to assert that many of the early contrived motions, although but recently adapted to meters, were made possible of success only by improvements in detail and machine processes; which, if measured upon the basis of practical results, are little if any inferior to the generic principle involved; for while it is true that Pappenheim, Galloway, Dakeyne, Ericsson and others had given us the thought, it is equally true that the seed had lain dormant until, with proper conditions of time and circumstance, they were exhumed, refashioned and transposed to a state of perfection and efficiency not dreamed of by the originators.

Reference has been made to the degree of accuracy which is regarded as sufficient in meters to be used in ordinary domestic service. This of course is simply an opinion. As a matter of fact, however, all of the principal meter makers in the United States regularly turn out meters capable of registering within probably 1 to 1.5 per cent. of accuracy, between ranges of 100 per cent. in rates of discharge, and having a guaranteed "life" of from five to ten years. For close scientific work, as in boiler testing, the writer has frequently provided meters calibrated to

practical accuracy, and that, too, with a range of 50 per cent. in rate of discharge.

The recent reductions in the cost of aluminum have made this an inviting material for experiment as a possible substitute for hard rubber, but the results so far, in the writer's experience, have been quite unsatisfactory; as it was found that aluminum when contained in ordinary brass castings with water, whether static or flowing, developed galvanic action, that resulted in producing a deposit which accumulated upon the aluminum element, to such an extent as to soon interfere with the proper mechanical operation of the parts. That this was in fact due to galvanic action was apparently proved by removing the brass element and immersing the aluminum in water contained in an ordinary porcelain basin, when no deposit took place. These experiments were tried with the regular commercial grade of metal, about 98.5 per cent. pure, made by the Pittsburgh Reduction Company, and also with aluminum made by the Castner process, claimed to be about 99.5 per cent. pure. If anything, however, the purer metal appeared to be the more readily acted upon. Mr. Alfred E. Hunt, M. Am. Soc. C. E., is now investigating the subject, and will undoubtedly soon be in a position to describe the cause thereof, even if unable to designate the remedy; neither of which is within the scope of the writer, who simply pretends to here note the results of his observations in this particular.

As to the future demand for meters, it is as certain to increase as it is certain that the water supply is more nearly a constant quantity than the consumption thereof; consequently, the trend of the purchaser on the one side is to the lowest price for equal value in performance; while on the other side the effort of the engineer is to meet the demand, which inevitably must come, under these premises, from the system involving the minimum of material and workmanship.

As to the arguments that may be made, pro and con, in respect of the different types of meters, they will of necessity partake of the color and form peculiar to the individual. Thus, a peculiarity has been observed in patent practice, common with applicants who appear for the first time in this class, in that their invention is liable to be entitled a fluid meter. It is not enough that it is a water meter of the first order; but it must include all fluids as well. However, with increased experience it is usual to find that a more specific title is accepted, even when, in patent office phraseology it is also guarded by the words "Substantially

as specified." And so as to the arguments for types of meters; if confined to the conditions to be met in service the specific features of advantage or disadvantage, one way or the other, may be grouped in small space.

In respect of the reciprocating piston meters, particularly of the duplex type, they are reliable, fairly accurate and durable. On the other hand, the objections to be urged are that they are bulky, heavy, limited in maximum capacity, cause pulsation and often water-hammer in the pipes, usually present serious obstruction to the flow, and are expensive to manufacture. Bulk and weight are objected to because of increased expense and trouble in shipping and setting; pulsations in mains are annoying, particularly in house service, and are also deleterious to fittings; obstruction to the free discharge, where the effective head is already barely adequate, may mean much loss of time to the householder or to the manufacturer.

In respect of the positive rotary meters, they are also reliable, usually very accurate and durable. About the only objections that can be urged against them are, that when hard rubber is employed in their construction, they are liable to damage from hot water and may, to use a waterworks phrase, occasionally "go to sleep;" that is, if obstructed by sand or scale, permit a small flow to pass without registration. On the other hand, they are compact, light, inexpensive to transport, convenient to handle and set, may usually be constructed of bronze composition instead of iron, and are susceptible of manufacture at a low price.

In regard to the inferential rotary meters for small services, it is probable that there is no future for this style of meter in the United States, and also that their use abroad will decrease. The reason for this opinion is that the positive rotary meters can now be sold at a price approximating that of the inferential; and as the positive class of rotary meter is more accurate and durable than the inferential, the controlling advantages heretofore possessed by the latter—compactness and low price—are neutralized.

As to the type of the meter of the future, for the smaller services, it seems clear, therefore, that the light, compact, and low-priced positive displacement "rotaries," whose efficiency is now well established, are certain to supplant the more cumbersome and expensive meters of the reciprocating piston type, as also the less accurate and durable meters of the inferential system. Has the meter of the future yet been born

in thought or molded to form? The author does not believe that it has; but of this he feels confident, namely, that if it shall hope to successfully cope with the already practically satisfactory and accepted meters now in the field, pushed by efficient and energetic rival companies, having ample capital, it must not only be better in quality and performance but must also be capable of being manufactured and sold at a still lower price. And "there's the rub;" for while the author would not hesitate to contract, under a suitable retainer, to regularly bring out a "new" meter, say every successive sixty days for a year, which would in all cases be an accurate and reliable water-measuring machine, he would yet require the days to be changed into months if this single additional clause were inserted in the bond: "The meter to be commercially successful."

As to what should be the controlling and altogether essential features of the perfect meter—for it is yet to be fashioned, and is as far from accomplishment as a perfect engine or a flawless rod of steel—he would say that if Emerson had been a hydraulic engineer, this problem must have been in his mind when he wrote: "If the right theory shall ever be discovered, we shall know it by this token: that it shall solve all riddles." And with diffidence the author presumes to say that for this riddle he can supply the token, but the riddle is still for you to solve. The token is this: The perfect meter must be frictionless.

In conclusion, the author has but this observation to submit, which is especially, and with all good wishes, offered to the new aspirant who may come forward expecting to win fame and fortune from the successful working out of this problem; it is the matchless aphorism of Lord Bacon, and should be ever kept in mind: "If experiments are not directed by theory, they are blind; if theory is not sustained by practice, it is deceiving and uncertain." For in no other branch of engineering of which the writer has any knowledge is there required a closer matching, so to speak, of sound theory with good practice than in the development of a modern commercially successful water meter.

DISCUSSION.

E. Kuichling, M. Am. Soc. C. E.—The subject of water meters has been presented in a most interesting manner by Mr. Thomson, and the memoir is worthy of careful consideration from all who are concerned in the management of municipal water-works. While it may be conceded that the only rational way of charging and paying for water consumed by individuals or corporations is by meter measurement, yet the present cost of these measuring devices and their maintenance is generally regarded as being altogether too large to render their extensive introduction expedient in our large cities. Many water-works officials would doubtless cheerfully recommend, and perhaps strongly urge, the adoption of meters for all classes of consumers, if they could obtain reasonably accurate, sensitive and durable machines at somewhat lower prices than appear to prevail at the present time; and it is mainly in consequence of existing prices, which the public regards as too high, that all efforts to introduce a general meter system have, in the majority of cities, met with determined opposition.

The essential qualities of a water meter are accuracy, sensitiveness, and durability; and to render them more popular than at present, the quality of cheapness should also be added, or perhaps even be placed first. It is gratifying to learn from Mr. Thomson's paper that much might be accomplished in all of the other directions by manufacturers and inventors of such devices, if only some reduction could be made in the unduly high standard of accuracy which seems to be required in most localities. Under such circumstances, it is obvious that some fair limit of accuracy should be established, in order that the advantage of largely reduced initial cost and greater durability may soon be realized.

In lowering the limit of accuracy, however, the standard of sensitiveness must not be affected, but, on the contrary, should rather be increased than diminished. Leakage of fittings and waste by small streams or driblings flowing constantly, usually give rise to greater consumptions than the legitimate use. The writer has often measured the leakage from a defective faucet, ball-cock, and closet-valve, in households and places of business, and found that a discharge of from 150 to 400 gallons per day from a single fixture rarely excites notice on the part of the inmates, and that a request to repair such fixture is regarded as grievous oppression. A loss of several hundred gallons per day by continuous leakage, in a household where the legitimate use is actually much smaller than this amount, is a circumstance of frequent occurrence where an efficient system of house to house inspection is not enforced. In the case of a town where the total daily per capita supply is only 50 or 60 gallons, it is obvious that all such insidious sapping of the distributing pipes will seriously impair the efficiency of the works, and perhaps even lead to a water famine.

The remedy usually prescribed in such a community is general metering; but here we are at once confronted with the fact that many of the meters in the market either fail to register such small streams, or else that the power required to overcome the friction of closely-fitted parts is greater than is tolerable. Thus, in testing recently a certain meter, which was to be set in a small private dwelling containing a family of only five persons, it was found that the machine would pass leakage at the rate of 1 500 gallons per day without registration, while on the other hand, when the discharge was increased to a rate of 5 000 gallons and more per day, the dial record was practically correct. Manifestly, such a meter is perfectly useless as a waste-preventing device, since it not only permits a leakage of at least five times the legitimate water consumption of the entire family to pass absolutely unrecorded; but it also enables any shrewd consumer to obtain vastly more than a copious supply without equitable charge, by simply allowing a small stream to flow continuously into a suitable distributing cistern. If it be assumed that a daily per capita consumption of 40 gallons is a reasonable quantity, then an average family of five persons should use 200 gallons per day, all of which would ordinarily be drawn at a rapid rate from the fixtures, and would therefore probably be recorded by almost any meter; but if only a single fixture in the dwelling were leaking at the rate of 200 gallons per day, how many meters would reveal the fact on their dials, especially after having been in service for a few years ?

It is very justly said that water is not sold to consumers by the drop or driblet, but by the gallon or the cubic foot, as the smallest practicable unit. To this statement no one should take much exception, but it is also fair to take it literally. Let the meter be sensitive enough to record, even with wide margin of accurancy, the fact that hundreds of gallons pass through it in the course of a day, and it will be a far more useful instrument to the community purchasing it than one which will exhibit marvellous accuracy in the measurement of comparatively large streams, and yet allow small flows to escape without detection. The percentage of error in registration of a meter is therefore a matter which should be sharply defined, and should be made dependent upon its sensitiveness. A loss of 10 per cent. on the above-named legitimate use of an average family would amount to only 20 gallons per day; and a loss of 25 per cent. on a leakage of 200 gallons per day is only 50 gallons, so that a total loss of 70 gallons, in this case, would be only about 17 per cent. of the whole quantity passing through the meter; whereas if the legitimate use alone were recorded with absolute accuracy, while the small leakage escaped registration, the loss would be 50 per cent. of the whole draught. In the case of meters intended for family use, which in a general meter system would vastly outnumber all other kinds, it is, therefore, essential to make sensitiveness a paramount feature, as well as durability and cheapness. The same requirements may also be made

for meters, for the majority of manufactories and places of business in a community; and in the comparatively few instances where the draught is constantly large, special meters which give a high degree of accuracy can easily be applied.

In conclusion, therefore, it may be stated that a thoroughly serviceable meter should have great sensitiveness, but need not have a very high degree of accuracy. How these qualities can be combined with durability and economy is a question whose solution is left to the skill of inventors.

JOHN THOMSON, M. Am. Soc. C. E.—In reply to Mr. Kuichling: First.—I disagree with his conclusion that "existing prices" for water meters have had much to do with the matter of their general application; but with the express proviso hereto that this opinion refers to municipalities and not to private water companies. Many instances might be cited in which the application of a few thousands of meters, even at the highest prices that have ever obtained in this market, would have resulted in the saving of millions of money expended in duplicating conduits and pumping plants, and in paying for the wanton waste of the many. I presume to submit that if 50 000 water meters had been offered to the City of New York at, say, $5 each, with a guarantee of their satisfactory performance, under bond, for a term of ten years, that the new aqueduct would have been built just the same, although the original supply, by such means properly conserved, would in all likelihood have been ample for such a term. The fault with the commercial reasoning of my friend Kuichling is that he is focusing his argument with the eye of an engineer alone, with no allowance for errors of parallax and the aberrations due to atmospheric disturbances. Nevertheless, the fact is not to be lost sight of that very considerable reductions in the prices of meters have been made within the past few years, the cause of which, in my opinion, is not due so much to the demand for lower prices as to the increased necessity for more meters. This has been a forced exigency; it is not that their use was right, proper and equitable, but that if not employed the use of water in such instance would be greater than the supply. In any market, as well as in that of the meter market, this might properly be designated a case of forced demand, originating with the consumer. Now, with this has concurrently grown a greater incentive to manufacture, which, in turn, has engendered increased competition, resulted in improved designs, caused the production of larger quantities, thereby reducing the cost of manufacture; and I believe that to these exterior causes, added to those previously herein set forth, are chiefly due the present prices of our commercial water meters. And if this is true of the past, it will probably be equally applicable to the future.

But I am glad to drop this phase of the subject and take up its alternate side. As to lowering the standard of accuracy, this is a suggestion

which I made with considerable hesitancy, as it might be taken as appearing to advocate a step backward. Nevertheless, I am of the opinion that, in at least the great majority of instances in which meters would be comparatively largely employed, the fact of whether or no they indicated at comparatively low rates of flow would bear but a very small relation to the total advantage and revenue derived therefrom. The point made by Mr. Kuichling to permit a lowering of the standard of accuracy, while yet retaining the element of sensitiveness to such a degree as to at least indicate a portion of the entire quantity at the lowest practicable rates of discharge, is somewhat in line with the results obtained in the use of the low-priced inferential meters, a type but little known in this country. This is a condition of operation, however, which would probably be very difficult to obtain, if at all, in the existing types of positive displacement meters; although the condition opposite to this could, I apprehend, more readily be reached; that is, to cause the meter to indicate in excess of the proper quantity at the lower rates of discharge. With many private water companies, this would be desirable; but whether or no it would be generally acceptable is a question I am not prepared to answer. The argument in favor of such a condition is that when it is known the meter will over-indicate on "driblets," then a leak, ordinarily passed as of no consequence, would have the first attention. On the other hand, the proposition is freqently held that the moral effect of placing a meter in the circuit of the service pipe is in itself all that is necessary to obtain the desired result, and that if only a meter casing, provided with a register but having no internal mechanism whatever, were generally applied in cities having the highest per capita rate of consumption, this, in itself, would bring the rate down to reasonable dimensions. In such a predicament, the mean value of the extremes may be the wiser to adopt. However, it is my judgment, based upon some little personal contact with water-works engineers and superintendents, that it is very unlikely any lowering of the present standard of accuracy would be accepted; while it is equally likely that the necessity therefor will grow less and less as the administration of water-works practice improves.

With probable interest and benefit, the mechanical difficulties might now be indicated which lie in the way of putting into practical effect Mr. Kuichling's idea of partial indication at low rates of delivery; but I fear such might not be clearly presented without the employment of additional illustration, which present opportunity makes it impossible to prepare. So, too, in respect of my suggestion to reverse the condition proposed by Mr. Kuichling, whereby to obtain over-registration at low rates of flow; as a discussion thereof would, at least, trend towards the class of proportional water meters, purposely omitted from the main body of this paper, and which I have had in mind to treat separately at some future time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

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489.
(Vol. XXV.—July, 1891.)

EXCESSIVE RAIN-FALLS CONSIDERED WITH ESPECIAL REFERENCE TO THEIR OCCURRENCE IN POPULOUS DISTRICTS.

By CAPTAIN R. L. HOXIE, M. Am. Soc. C. E.

READ JULY 2D, 1886.

WITH DISCUSSION.

PART I.

In the practice of hydraulic engineers it is necessary, at times, to provide absolute protection against the most violent storms that can occur in a drainage area, whether of a large or small extent; it is of primary importance to know what such a storm is.

PRECIPITATION OF RAIN.—Meteorological records are ordinarily made from observations taken at stated intervals, rarely less than four hours, and the detailed phenomena of storms are seldom recorded. The rate of precipitation given is the mean rate of this interval, and the maximum rate has been little regarded. The importance of this maximum rate increases as the area of drainage diminishes, since a heavy rate of short duration may seriously affect a small area. The following table has been com-

Table showing Observed Precipitation of Rain-fall in Inches, during Violent Storms.

Locality and Sources of Information.	Duration of Rain fall in Minutes.										Duration of Rain-fall in Hours.																
	1	5	8	15	20	22	25	30	40	45	1	1.25	1.50	2.00	2.33	2.50	3.00	3.25	4.00	5.00	7.50	14	24	30	36	48	56
	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.
Great Britain ..... (I.)	0.2	0.75		1.0				1.8		2.5	3.25			3.60			4.00										
Washington, D.C.....(II.)		0.52	0.47	0.75	0.74	0.97	1.78	1.14	2.0	1.14		1.25						4.92 (a)									7.92
Nottingham, Eng.....(III.)								1.16																			
Birmingham, Eng. ....(III.)								1.94																			
New York .....(III.)																				4.00							
Providence .....(III.)				0.90	1.42						3.15																
London and vicinity ...(IV.)				0.50				2.00			4.00		6.00		3.30												
Paris ....(IV.)					1.61			1.26	0.64		1.57																
Switzerland .....(IV.)		0.87						2.00																			
Geneva .....(IV.)																	6.38										
Munich .....(IV.)								2.00																			
Catskill.....(IV.)																					1.80						
Budapest .....(IV.)											2.55																
Marseilles ....(IV.)							1.57															13.0					
Genf.....(IV.)															6.39												
Kiel .....(IV.)			1.46		0.93														5.00							12.0	
New London .....(V.)																			5.00							12.0	
Norwich.....(V.)																	4.27										
Goffstown.....(VI.)																									12.35		
Canton, Conn. .....(VI.)														4.00													
Concord, N H. .....(VI.)																3.00											
Fitchburg, Mass..... (VI.)														2.5													
Norway, Maine.....(VI.)																							7.15				
Middletown .....(VI.)														6.00										8.90			
Philadelphia .....(VII.)																				9.00							
Norristown, Pa .....(VII.)																							32.				
Genoa .....(VII.)																											
Brooklyn .....(VIII.)							2.6		1.2		1.65																

piled from the sources indicated. With a single exception it is a chance record by occasional memoranda injected into systems of observation which ignored the importance of this record; as a consequence it covers a wide field in the search for information. The universality of the phenomena suggests the operation of some law of recurrence which cannot be deduced from the meagre record.

Sources of preceding Table.

(I.) From "Practical Hydraulics, a series of Rules and Tables for the use of Engineers," etc., by Thomas Box, London, 1876. Mr. Box says: "For town drainage and other purposes we require to know the maximum fall of rain during storms. We find that * * * the maximum fall of rain may be (as in table)."

(II.) From the meteorological records of the Signal Service, of the Smithsonian Institute, and of the United States Naval Observatory, Washington, D. C., including the continuous record by pencil curve of one self-registering rain-gauge at the Signal Office, Washington, D. C., from 1871 to 1885, both inclusive.

(*a.*) A letter from Professor Joseph Henry to Major G. K. Warren, Corps of Engineers, U. S. A., now on file in the Engineer Department, gives the record of a storm on the 29th of July, 1865, at Washington, D. C., in which 4.92 inches of rain fell between 3.15 P.M. and 6.30 P.M. The rain-fall during the storm was at an average rate of 1.510 inches per hour for three hours and a quarter; what the down-pours were can only be conjectured.

(III.) From report on sewerage in the City of Providence, by J. Herbert Shedd. The meteorological record kept by President Alex. Caswell, D.D., LL.D., from 1832 to 1873, in Providence, R. I., is carefully tabulated, so as to present valuable information; but the record of down-pours is limited to eight memoranda of Dr. Caswell, beginning in 1862. The rate in the case of each storm is obtained by dividing the total precipitation by the whole time of rain-fall, giving the mean rate for the entire period, and where the time of rain-fall was less than an hour it is reckoned as having been an hour in falling. This process eliminates all down-pours and greatly reduces the recorded number of violent storms having phenomenal rates of fall. However, these down-pours usually occur during the progress of storms giving a large total precipitation, and some idea of the frequency of down-pours may be derived from the

number of such storms. It appears from Dr. Caswell's record that during the forty-two years from 1832 to 1873, both inclusive, rain-falls exceeding 4 inches occurred at average intervals of 4.62 years, the longest interval being eight years, and the shortest less than one year, while rain-falls of 2 to 4 inches occurred every year but one (1846); the least number of such storms in one year being 1, the greatest number 7, and the average 3½. Mr. Shedd quotes from the annual report of 1847 of Mr. John Roe, surveyor of Holburn and Finsbury sewers, and author of the well-known "Roe's tables." "In August last the surveyor had occasion to report that 4 inches depth of rain had fallen in less than one hour in that month, a circumstance that cannot be too extensively known at a time when much sewer work is likely to be executed in this country; for having once experienced such a fall of rain it is right to expect and provide for the like occurrence in future."

(IV.) From report of Mr. Rudolph Hering to the National Board of Health, Washington, D. C., 1881, including Mr. Hering's citation of other authorities. This, with other and valuable information, was collected by Mr. Hering in the course of an extended tour abroad for this purpose. He says: "In only two towns (Zurich and Munich) did I observe rain-gauges of this nature (self-registering), and these had only recently been placed."

(V.) From a treatise on water supply engineering, by J. T. Fanning, New York, 1878. Mr. Fanning says in this connection: "When studies of local rain-falls are confined to mean results, neglecting the occasional wide departures from the influence of the general controlling atmospheric laws, the actions of nature seem precise and regular in their successions, and in fact we find that the governing forces hold results with a firm bearing close upon their appointed line. But occasionally they break out from their accustomed course as with a convulsive leap, and a storm rages as though the windows of heaven had burst, and floods sweep down the water courses, almost irresistible in their fury. If hydraulic constructions are not built as firm as the everlasting hills, their ruins will on such occasions be borne along on the flood toward the ocean."

(VI.) From a paper read before the American Society of Civil Engineers, at their Tenth Annual Convention, June 18th, 1878, by J. B. Francis, Past President, Am. Soc. C. E. Mr. Francis writes: "In the construction of catch-water reservoirs, mill-dams, culverts in railway and

other embankments, and many other works, provision has to be made for the maximum flow of water. In small areas this depends, to a great extent, on the maximum rain-fall in a given time." With the painstaking thoroughness of the "Lowell experiments," Mr. Francis has collected from a great number of observers the details of a rain-fall of about thirty hours' duration, and claiming only "a rough approximation" shows that the precipitation of—

					Square Miles.
6 inches or more				covered an area of	24 431
7	"	"	"	"	9 602
8	"	"	"	"	1 824
9	"	"	"	"	1 046
10	"	"	"	"	519
11	"	to 12.35	"	"	179

The isohyetal lines surrounding the central area of maximum precipitation with the symmetrical curves of the barometric wave. The observer at Concord, N. H., reports: "4 inches of rain fell between 12.30 P.M. and 2.30 P.M., October 4th." The observer at Goffstown, N. H., reports: "4.27 inches of rain fell between 11 A.M. and 2 P.M., October 4th." The observer at Fitchburg reports: "about 3 inches of rain fell between 11.30 A.M. and 2 P.M., October 4th." The air line distance from Concord to Fitchburg is about 45 miles, with Goffstown nearly on the line between them; if this distance be taken as the diameter of the area of simultaneous down-pour, it covered about 1 590 square miles.

(VII.) Trautwine. This author says: "The most destructive rains are usually those which fall upon snow, under which the ground is frozen so as not to absorb water."

(VIII.) From a report on the sewers of Brooklyn, N. Y., by Robert Van Buren and William E. Worthen (1878).

Maximum Flood-Flow.—In general, at any point in the course of the outflowing water, it will be found that the maximum flood flow from the drainage area under consideration is the most dangerous consequence of excessive storms, while the length of time such maximum flow continues will have more or less effect in augmenting the danger, or in extending its field of menace. When one of these great reservoirs of aqueous vapor opens its flood-gates by condensation and pours down its maximum rate of fall, it saturates absorbent surfaces if not already saturated, ponds upon flat surfaces to gather surface slope for discharge to the nearest channel, and then every point in the drainage area which the

precipitation covers becomes a fountain of drops, one following another in quick succession, and a fillet of drops proceeds from every point at once. They meet in rills and rivulets; they gather in the smaller streams, and flow in company to join with others in the larger branches. At any point in the course of the outflowing water, the fillet from the nearest point of the water-shed appears at once and continues to flow, while others join it in quick succession from points more and more remote, until the fillets from the rim of the water-shed, gathered in the tributary branches, are flowing together past the same point of the main channel. This is the maximum effect of the down-pour, when the grasp of the main channel encircles, at once, a fillet of drops from every point in the area it drains. The effect is quickly developed in the smaller streams, and moves down the main channel as fast and as far as the fillet of drops from the point most remote or that which is most belated by obstructions to flow, can travel while the down-pour continues. When the rain-fall slackens, the fillets dwindle, and when it ceases, the fillets entering the channel in front of the flood-wave run out to the end, one by one, through the channel, in advance of the coming wave, and fail to re-inforce it. The first effect, the conditions of which are simple, is the danger to be averted in the smaller drainage areas; the after effect, in which the crest of the flood-wave follows the slackening or cessation of rain-fall, is the dangerous effect in the larger basins, and is more complicated.

The maximum flood-flow from any drainage area is a function of many variables; the total precipitation of rain producing the flood; the mean rate of precipitation, the fluctuations in this rate giving periods of maxima and minima with their respective intensity and duration; the proportion of the drainage area covered by the precipitation, its location with respect to the point under consideration, its shape and topographical features, its general slope and distribution of hillside and valley with their relative slopes; its capacity for pondage in lakes, ponds and storm channels, and the character of its surface as affecting the question of absorption and evaporation over the entire area under varying meteorological conditions during the critical period between the commencement of the rain-fall and the arrival of the flood; as well as the friction in channels and other sources of detention of flow from the surface. In large river basins it is not practicable to ascertain the actual conditions of the problem; in smaller areas the difficulty diminishes until, in urban

districts of roofs and pavements only, these conditions may all be ascertained. As the natural surface changes by human agency, the rate of maximum flood-flow increases; trees and shrubs disappear and the surface flow is accelerated. Cultivated lands with tile drains and ditches further increase the rate of flow. When a drainage area lies within city limits, all pondage is obliterated; a surface more or less absorbent and retentive is replaced by smooth and non-absorbent roofs and pavements; eave-spouts, gutters and drains of quick discharge take the place of filled-up natural channels.

Maximum Flood-Flow in City Drains.—If a typical city area be assumed completely covered with roofs and pavements and provided with a completed system of drains, into which all the rain-water finds its way from roof or pavement at once, the drains having sufficient capacity to carry the maximum flood-flow, this question is greatly simplified. The falling water finds a smooth channel from the start which conducts it by the shortest line to the outfall. Modern hydraulic formulas give with close accuracy the conditions of flow in these channels. The meteorological records of the locality should indicate the storms which have occurred, and which may therefore occur again; absorption by surfaces of metal and stone, and evaporation into a saturated atmosphere during the period of maximum rain-fall, may be accurately computed. It has been shown that 94½ per cent. of a brisk rain-fall upon such a surface may be collected and discharged through a system of drains,* which was manifestly imperfect, while a natural surface of 6.6 square miles has delivered 98 per cent. under favorable conditions. The great work of main drainage of London brought prominently into notice the question of conveyance of rain-fall in city drains, the subject being incidentally investigated at that time by the eminent engineers connected with this project. In the plans adopted, this investigation has little weight. The several lines of intercepting sewers constructed to convey the sewage of the metropolis to a distant outfall, receive the dry-weather flow of the old valley sewers with a fixed and insignificant amount of rain, and pass the storm-flow through by overflows to be carried forward to the "Thames" in its former channels. In this way the question of storm-flow in the new works was practically eliminated; but the investigation evolved some interesting information, nothing more prominently than the prevailing ignorance of the

* June, 1885. The Iron gate sewer. Mr. W. Haywood in a discussion before the Institute of Civil Engineers, London, 1865.

probable storm-flow in city drains. In fact, the existing sewers of London were generally of insufficient capacity. Mr. Jebb in his "report on the drainage of the metropolis," 1854, says that many of the sewers were "unnecessarily large, still more of them too small, the consequence of which latter defect was that tenements were flooded and property destroyed to a great extent annually." Roe's tables, published in 1852, and Mr. Hawksley's formula, which gives quite similar results, became prominent, and were generally accepted as standard authority; they have continued in use to the present date.

Roe's Table, showing the quantity of covered surface, from which circular sewers, with junctions properly connected, will convey away the water coming from a fall of rain of 1 inch in the hour, with house drainage, as ascertained in the Holburn and Finsbury Divisions.

Inclination Fall or Slope of Sewer.	Inner Diameter, or Bore of Sewer in Feet.									
	2	2½	3	4	5	6	7	8	9	10
	Acres.	Acres.	Acres.	Acres.	Acres.	Acres.	Acres.	Acres.	Acres.	Acres.
Level	39	67	120	277	570	1 020	1 725	2 850	4 125	5 825
¼-in. in 10-ft. or 1 in 480	43	75	135	308	630	1 117	1 925	3 025	4 425	6 250
½ " " " " " 240	50	87	155	355	735	1 318	2 225	3 500	5 100	7 175
¾ " " " " " 160	63	113	203	460	950	1 692	2 875	4 500	6 575	9 250
1 " " " " " 120	78	143	257	590	1 200	2 180	3 700	5 825	7 850	11 050
1½ " " " " " 80	90	165	295	670	1 385	2 486	4 225	6 625		
2 " " " " " 60	115	182	318	730	1 500	2 675	4 550	7 125		

Level sewers have a capacity which diminishes with their length, the friction head being continuously formed at the expense of depth of flow in the sewer. In this respect the table is evidently unreliable. The sewers with stated slopes must be proportioned to take the maximum flood-flow in each case, else they would not suffice. Applying to the table, Kutter's formula, for the flow of water in circular conduits, we get the actual capacities of these sewers and find that they indicate a maximum rate of flood-flow (9 cubic feet per second), only 2-10 the mean rate of precipitation (1 inch per hour), for the smallest area (43 acres) of city surface, and a maximum rate of flood-flow (1 317 cubic feet per second), only 12-100 of the mean rate of precipitation for an area of 11 050 acres; yet modern observation has shown that from a small natural water-shed with natural channels, under certain conditions, the

maximum rate of volume of flow through the outlet channel may reach two-thirds of the average rate of volume of rain falling upon the gathering ground.* The storage reservoir of the Nagpoor Water Works, of British India, receives the drainage of 6.6 square miles of natural surface. On September 16th, 1872, there occurred a fall of 2.2 inches of rain in eighty minutes, a rate of 1.65 inches per hour. In one hundred and seventy minutes 98 per cent. of the total rain-fall was gathered into the reservoir, the ground being saturated when the storm occurred.† Nearly the same quantity (2 inches), has fallen in half the time (40 minutes), and a mean rate of 1.51 inches of rain-fall per hour has been sustained three hours and a quarter in the District of Columbia.

If such facts be considered with the conditions of city surfaces and swift flow in city drains, it will be seen why Mr. Roe's table and Mr. Hawksley's formula, have uniformly failed to give sufficient capacity to drains which must actually take the rain-fall. Plotting separately the curves of capacity for the several inclinations given in Mr. Roe's table, it is found that for equal areas the capacity required for an inclination of 1 in 60 greatly exceeds that for an inclination of 1 in 480, the difference increasing steadily to about 40 per cent. for an area of 6 250 acres. If these slopes be indicative of the general slope of water-sheds, as in valley sewers, this increase would harmonize with the more rapid discharge from steeper slopes; bnt the slope of 1 in 120 requires, by the table, a less capacity than either of the slopes of 1 in 160 or 1 in 240, and the capacity required for the two latter is nearly the same for equal areas. It is probable that Mr. Roe's twenty years' observations are vitiated by the same source of error that prevails at present. A natural surface gathers all the flow of rain-fall into its natural channels, but nearly all systems of town drains are imperfect as collectors of storm water. In a slow, gentle rain, the inlets may receive it all; in sudden down-pours they are unable to take it in; they are often proportioned to reject it, and a variable quantity escapes over the surface. This erratic variable discredits all gaugings of storm flow in sewers, and accounts for the extraordinary variations shown.

Mr. W. Haywood says, in 1865 : "Of a rainstorm of 0.54 of an inch in five hours, in June, 1858, there was delivered into the Iron-gate sewer, which drained an area entirely paved and built over, as much as 94½ per cent. of the total rain-fall, and that, of all the storm gaugings he had

* Fanning.

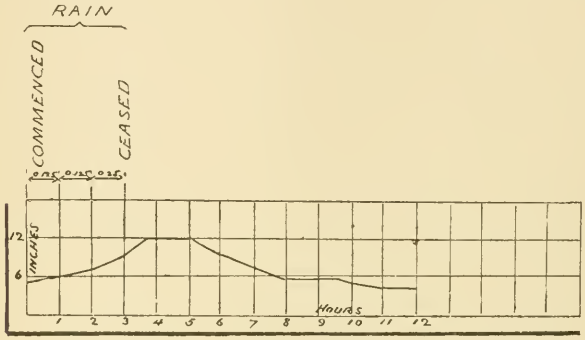
† A. R. Binnie, M. I. C. E. Proc., Inst. C. E., Vol. 39, 1874–75.

made, was the greatest percentage of rain-fall he ever knew to be discharged by a sewer. In August, 1858, with a rain-fall of 0.48 inch in one and two-thirds of an hour, he found only 78 per cent. of the total quantity discharged in the Iron-gate sewer." It will be noted that in a moderate rain of 0.54 of an inch in five hours, 94½ per cent. found access to the drains, but in a violent rain of 0.48 of an inch in one and two-third hours, only 78 per cent. got in. Absorption and evaporation had the better chance to deplete this volume in the first case. Had the sewer drained a basin without surface outlet the ratio of loss in the second case would, doubtless, have been less than in the first. When such gaugings pertain to sewers having a fixed regimen, *i. e.*, into which no larger proportion of flood-water will ever be permitted to enter, they give useful information for that sewer and for no other. Unfortunately the deductions from the record of these sewers have been promiscuously applied to others—even to sewers and drains which, draining basins without surface outlet, were compelled to carry off the total rainfall. It will be noted that the distinguished engineers connected with the London main drainage works did not go elsewhere for statistics, but gauged the actual flow of the old sewers which they had to deal with.

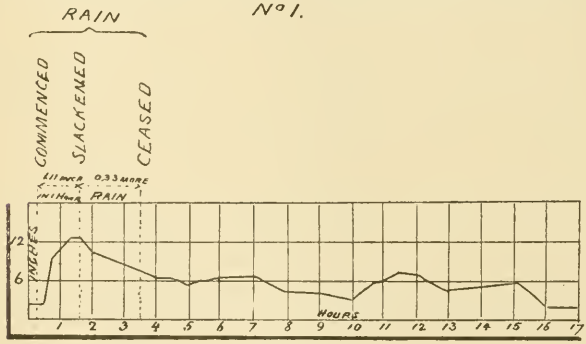
The published records of sewer gaugings are in other respects unsatisfactory. While they show that urban surfaces are practically impervious, and evaporation insignificant in violent storms, the rate and duration of the maximum flood-flow has received but little attention. Instead of this, the time in which a certain proportion of the rain-fall passes off, is used to deduce a mean rate of flow.

In Mr. Shedd's report on the sewerage of Providence, two diagrams are published from a report of Mr. Roe, in 1852; these show, conspicuously, the time of appearance, volume, and duration of the maximum flood-flow in two cases—incidentally, however, as they are intended to "illustrate the length of time required for the water of a rain-fall to reach the sewers." In a rain-fall of ½ inch in three hours the maximum effect was developed in three and three-quarter hours, but the effect of the storm continued for twelve hours. In a rain-fall of 1.11 inches in one hour, followed by 0.33 in the next two hours, the maximum effect was developed in about three-quarters of an hour, before the first heavy fall had slackened, but the storm water continued to flow for sixteen hours. The maximum flood-wave developed in the second instance was the dangerous maximum resulting from a simultaneous flow from all points

of the drainage area during the maximum rate of precipitation. It would be interesting to know how far from the head of the system it traveled before the slackening of this rate occurred. Of these diagrams, Mr. Shedd says: "Unfortunately Mr. Roe does not give the rate of inclination nor the character of the surface from which the water flowed, nor other facts which it would be useful to know; but I infer that the diagrams are given as fair samples of what may be expected in a district like this, where the extreme difference of elevation is about 400 feet in a distance of 5 miles."



No 1.



No 2.

An average velocity of 6 feet per second in a completed system of drains would carry the maximum flood-wave due to a down-pour of one hour's duration to a distance of 21 600 feet from the most remote point of the rim of the drainage basin. If the main drain lie symmetrically in the valley of its drainage area, and the latter be semicircular—a shape to which the upper portion of such areas frequently approximates, this distance of 21 600 feet, say 4 miles, is the radius of a semicircular area

of 50 square miles, from every point of which the maximum flood-wave 4 miles from the rim at the expiration of the hour, would be drawing a contribution. Long after the down-pour was over, the mean flow would be passing the outlet, and long after that the miles of gorged drains would continue slowly to empty themselves.

Of the rain-fall of October, 1869, which he investigated so thoroughly, Mr. Francis writes: "At Concord, N. H., out of the whole rain-fall of 7.40 inches, 4 inches fell in two hours on the afternoon of the second day of the storm, and undoubtedly in many other localities there was a like down-pour, and in some places even more. This inequality in the rate has an important bearing on the effect of such a storm on small areas. The long-continued and heavy rain must have filled up the ground and usual storage reservoirs, and, being followed immediately by a fall of rain which, of itself, would make a flood, a great part of the latter in a hilly country must have passed at once into the streams, and the flow in the smaller streams, for some period of time, must have been nearly that due to the rate at which the rain was falling on the water-shed supplying them."

On June 28th, 1881, a rain-fall of 2.68 inches occurred in Washington, D. C. It included a precipitation of 1.78 inches in twenty-five minutes, a rate of 4.23 inches per hour. The New York Avenue intercepting sewer had just been completed. It drains an area of 436 acres, of which 200 acres are paved with asphalt and closely built over; 80 acres are open park, and 156 acres are sparsely built upon but mainly paved with asphalt. The inlets for storm water were imperfect, and in heavy showers a large quantity of water flowed over the surface of the streets across the line of the intercepting sewer. Under these circumstances the sewer was gorged, the water rising high above the crown of the arch in the manholes. It was computed by Kutter's formula to carry 2 inches of rain-fall per hour, or about 2 cubic feet per acre per second. In this instance the rain preceding the down-pour had saturated what little absorbent surface existed, and the down-pour itself must have been thrown off with hardly appreciable loss. Allowing five minutes to develop the maximum surface flow toward the drains, twenty minutes of down-pour remained, during which a mean velocity of 5 feet per second would carry the maximum flood-wave from the most remote point of the crest of the water-shed to the outlet. The mean velocity in these drains is probably about 7 feet per second, and there

can be little doubt that had the main sewer possessed the requisite capacity and suitable inlets, its flow must have been for some period of time nearly that due to the rate at which the rain was falling on its water-shed of 436 acres.

In 1884, a self-recording gauge was placed at a point in this sewer where it is circular in section, 8 feet in diameter, and carries the drainage of about 200 acres paved with asphalt and well built up. After the gauge was placed in position a storm occurred of $\frac{1}{2}$ an inch in fifteen minutes, a rate of 2 inches per hour. The flow in the sewer rose almost immediately after the rain began, and fell to its normal level within a few minutes after the rain ceased; its maximum height was $5\frac{7}{10}$ feet, or about three-quarters the capacity of the sewer. A second storm occurred June 28th, 1885. The record of the sewer-gauge tallied exactly with the rain-gauge of a few minutes earlier. At its maximum period the rain fell for thirty-seven minutes at the rate of 0.9 of an inch per hour. The sewer-gauge rose to a height of 3.7 feet, giving about 0.47 of the capacity of the sewer, and indicating no loss whatever by absorption or evaporation during the time of maximum flow.* The rain-gauge in each instance was that of the Signal Office, situated within this water-shed near its western border, but outside of the 200-acre paved area to the eastward. If the distance from the rain-gauge to the crest of the water-shed on which the maximum effect was gauged, be taken as the radius of the area covered by these storms, this area was about 4.7 square miles. The storm of June 28th, 1881, showed the effect of violent precipitation over at least 10 square miles. Not unfrequently a statement of excessive storm-flow from such a water-shed is met by a comparison with the summer flow of rivers, upon the assumption that the disproportion in areas discredits the observed fact. If it be remembered that the flood-flow of some navigable rivers is from 200 to 1 500 times their least flow, the comparison has no significance.

Theoretical Consideration.—Notwithstanding the complicated nature of this question, the absence of sufficient data, and the imperfection of records, a great number of formulas have been devised to solve the problem. The comparison of these is instructive.

* Reports of Commissioners, D. C., for 1881-84.

TABLE of maximum flood-flow in cubic feet per second from drainage areas of 64 acres to 6 square miles, computed by various formulas.

Square Miles of Area Drained.	DISCHARGE IN CUBIC FEET PER SECOND.												
	Craig.	Dredge.	Dick-ens.	Fan-ning.	Kirkwood.			Hawksley.			Burkli Zeigler.		
.1	282	222	147	29	18	21	22	17	35	41	61	135	161
.2	469	353	247	52	35	40	41	30	63	73	103	228	271
.3	625	462	334	73	51	58	60	42	87	102	139	309	367
.4	763	560	415	93	65	75	78	53	110	129	173	383	455
.5	889	650	491	112	79	93	96	64	132	155	205	453	538
.6	1 007	734	562	131	93	108	112	74	153	179	234	519	617
.7	1 116	813	631	148	107	124	129	84	174	203	263	583	693
.8	1 221	889	698	166	120	142	145	93	194	228	291	644	766
.9	1 321	962	762	183	133	157	162	102	213	250	318	704	837
.10	1 416	1 032	825	200	147	172	178	112	232	272	344	762	906
.20	2 225	1 638	1 388	356	274	323	333	195	408	476	579	1 281	1 523
.30	2 886	2 146	1 881	500	390	464	479	269	565	662	784	1 736	2 064
.40	3 463	2 600	2 334	635	503	597	623	340	711	837	973	2 153	2 561
.50	3 9-6	3 017	2 759	765	613	734	787	405	850	997	1 150	2 546	3 028
.60	4 468	3 407	3 163	890	720	863	891	468	985	1 158	1 319	2 919	3 471

NOTE.—Where more than one variable enters the formula, the values assumed are indicated in the description following, and are taken so as to give such uniformity of conditions as the formulas will admit. The three columns given to each of the last three formulas correspond to slopes of $\frac{1}{480}$, $\frac{1}{20}$ and $\frac{1}{10}$ successively.

Mr. Craig's formula is the following:

$$D = 440 \times B \times N \text{ hyp. log. } \frac{8L^2}{B}.$$

In which D represents the discharge in cubic feet per second; L the extreme length, and B the mean breadth of the drainage area, and N an arbitrary constant varying from 0.37 to 1.95, which depends upon the meteorological and topographical conditions of the locality; the least value of N is assigned to very flat river basins. In applying the formula to river basins, the flood discharges of which have been observed, and from which the values of the arbitrary constant are deduced, the author gives no example of an area less than 120 square miles, but the mathematical deduction of the formula applies equally to the smallest area. The meteorological conditions considered being those which apply to large river basins in India, the principal factor is the mean intensity of prolonged rain-falls there. The greatest value of the mean intensity of a prolonged rain-fall in India may be fairly compared with the mean intensity of the down-pours in this climate, which are of shorter duration, and are destructive in comparatively small areas.

In the table $L = \sqrt{\frac{5\,M}{4}}$, $B = \sqrt{\frac{M}{5}}$, M = area in square miles, and $N = 1.16$. The Dredge formula resembles Mr. Craig's in introducing the shape of water-shed:

$$Q = 1\,300\,\frac{M}{L^{\frac{2}{3}}}.$$

In which L is the length of water-shed, and M the area in square miles. In the table $L = \sqrt{2\,M}$.

Colonel Dickens' formula is the well-known type of formulas, having but a single variable, the area:

$$D = 825\,M^{\frac{3}{4}}.$$

In which D represents the discharge in cubic feet per second, and M the area of drainage basin in square miles. Mr. Fanning's formula is of this type. In presenting it, the author says the recorded flood measurements of American streams are few in number, but upon plotting such data as is obtained, we find their mean curve to follow very closely that of the equation:

$$Q = 200\,(M)^{\frac{5}{6}}.$$

In which M is the area of water-shed in square miles, and Q the volume of discharge in cubic feet per second from the whole area. All of the foregoing apply to the natural surfaces of river basins.

In the Burkli Zeigler formula:

$$Q = R\,C\sqrt[4]{\frac{S}{A}}.$$

The arbitrary constant C represents the character of surface, the highest value 0.75 being for paved streets and densely built up areas, the lowest value 0.31 for macadamized streets; R is the average intensity of rain during the period of the heaviest fall in cubic feet per acre per second (for Central Europe this rate was found to be from $1\frac{3}{4}$ to $2\frac{3}{4}$ cubic feet per acre per second), S is the general grade of area per thousand, Q is the water reaching sewers in cubic feet per acre per second.*

In this formula Q increases with R, C and S, and diminishes as A increases. The condition of equal delivery per acre from all areas, large or small, is that $S = A$. The formula takes no account of the shape of water-shed.

* Report of Mr. Rudolph Hering to National Board of Health, June 28th, 1881.

In the table $R = 3$ cubic feet, $C = 0.75$ and $S = 2.08$, 50.16, and 100, successively.

Mr. Kirkwood's formula is:

$$D = \left(\frac{N^2}{5804S}\right)^6$$

in which D is the diameter of sewer in feet; S the sine of inclination, and N the number of acres of city area which this sewer will drain. In the deduction of his formula Mr. Kirkwood assumes a maximum rain-fall of 1 inch per hour, and a maximum outflow of one-half of this for all areas, large or small. He uses Prony's formula to deduce an expression for D, which he then modifies so as to make the results conform to Roe's tables. Q is obtained in the above table by using Kutter's formula, with values of D and S obtained from Kirkwood's formula, and $S = \frac{1}{480}$, $\frac{1}{20}$ and $\frac{1}{10}$ successively.

Mr. Hawksley's formula is:

$$\log.\ D = \frac{3 \log.\ A + \log.\ N + 6.8}{10}$$

in which D is the diameter of sewer in inches, required to carry off the outflow from 1 inch of rain per hour; A the number of acres to be drained, and N the length in feet in which the sewer falls one foot. Both the formula and the tables are designed exclusively for city or suburban surfaces. Values of Q in the table are deduced by using Kutter's formula with values of D and N obtained from Hawksley's formula, and $N = 480$, 20 and 10 successively.

The formulas of Kirkwood and Hawksley, and Roe's tables, are functions of two variables only, the area of drainage and slope of conduit; each is independent of slope and other characteristics of water-shed, except as this slope may conform to that of the conduit, as in valley sewers; and all rest upon the assumption of a maximum rain-fall of 1 inch per hour. It will be observed that they give values of maximum flood flow for paved areas, which are far less than the actual observed flood flow for natural surfaces. The conclusion is inevitable, either that the maximum rain-fall assumed is very much too small, or that the formulas resting upon sewer gaugings extending over a period of twenty years, represent the flow of rain-fall that found access to the sewers, and not the maximum flow that will obtain under circumstances which require them to take it all. It seems probable that they are affected by both sources of error. These formulas and tables have been

extensively used in designing sewers and drains, with the uniform result of disaster whenever the drains have been actually required to perform the service for which the formulas and tables declare them competent.*

Applied to the gauging of New York avenue sewer, in Washington, D. C., in 1884 (see *ante*), empirical formulas give the following discordant results:

CAPACITY in cubic feet per second required in a circular sewer, draining 200 acres of paved surface, to carry off the rain-fall. The slope of the sewer being .002 and the general slope of water-shed being .008:

RAIN-FALL.	ROE'S TABLES.	HAWKSLEY.	KIRKWOOD.	BURKLI ZIEGLER.	ACTUAL MAX. FLOW AS GAUGED.
.5 inches in 15 minutes	36.3	43.2	51.7	137.6	300
.55 inches in 37 "	36.3	43.2	51.7	61.9	180

In the first case the maximum rate of outflow, as gauged, was three-fourths the rate of downpour (2.0 inches per hour). In the second case the maximum rate of outflow, as gauged, equaled the rate of downpour (0.9 inches per hour). Either the duration—fifteen minutes—was not quite sufficient to develop the maximum effect of the first storm at the point where the gauging occurred, or its violence caused the escape of a small proportion over the surface. It will be noted that the formula of Burkli Ziegler gives for this water-shed a maximum rate of outflow $33\frac{1}{3}$ per cent. of the maximum rate of any rain-fall, while the gauging in the second instance shows no difference in these rates. If the maximum rate of rain-fall be assumed at 2 inches per hour, the formula gives a capacity of main drain about one-third the capacity of the existing drain which has been gorged in actual service.

Professor J. B. Johnson has recently been arguing with much force the sheer impossibility of expressing by means of formulas the conditions of flow of a river at any point, comparing the problem with that of the determination of the instantaneous relation existing between the pressure of the piston of a crank-engine, the resistance to motion, and

* The formulas of Kirkwood and Hawksley apply to a fall of only 1 inch in one hour, and deem this sufficient provision for storms; but Mr. Roe has another table for 2 inches of rain-fall per hour, and has recommended that provision be made for a storm of 4 inches in one hour.

the velocity of the flywheel.* It requires no argument to point out the impracticability of such generalizations as are involved in empirical formulas for maximum flood flow. They are useful, as each suggests its relation to the cases to which it is made to conform, and so widens the field explored; but there is no legerdemain in hydraulics which can solve a problem of twenty independent variables by the use of one or two. These empirical formulas "which can only be applied with confidence within the limits of the experiments on which they are based," are too often mistaken for devices by which a few hours of elementary computation may suffice instead of laborious thought and thoroughness of investigation.

PART II.

Expediency of Making Adequate Provision in Systems of Drainage for Excessive Storms.

The necessity for such provision is well recognized in all hydraulic works, the destruction of which by flood would involve loss of life or serious damage to property. The primary object of the earliest sewers was such provision for storms. When their use as sewers developed the fact that their shape made them unsuitable for this purpose, "the separate system" of small sewers arose and has ever since taunted the "combined system" with its former size. The old storm sewers of great size and with flat inverts were certainly not adapted to the conveyance of sewage; but given sufficient capacity, their original sin, and every one of them might have been provided with an invert as in the Paris main sewers, which would carry the dry weather flow to the best possible advantage and with absolute precision of adjustment to its fluctuations, while the storm capacity remained intact. The expediency of so doing is another question. Mr. Rawlinson, the apostle of the separate system, had a profound respect for the magnitude of the task imposed upon the combined system by the duty of conveying storm water. In his "Suggestions," he says:

"Hydraulic tables may be of service, but they must be applied under the discrimination of practical knowledge, or they may prove to be more misleading than useful. For main sewering the engineer may profitably consult tables of rain-fall, or areas, and of the flow of water from the site, and it will be quite in order for him to do so; but if he

* Proceedings of the American Association for the Advancement of Science, 1885.

attempts to apportion sewers so as to be of capacity to receive and remove the flood water 6 to 10, or more, feet below the surface, he will commit a serious and costly blunder, as heavy falls of rain cannot be dealt with in such manner. A rule-of-three sum, made up of 'area, rain-fall and dimension of sewers,' cannot safely be acted upon. Heavy falls of rain must pass at and over the surface as during previous times, or special drains must be used if they exist, or must be provided for if they do not exist; and the sewers must be confined in subsectional capacity, so as to be equal to the removal of waste water from houses and manufactures. In large towns there may be exceptions; and then, as in London, flood water overflow channels into natural streams may have to be formed."

In this he concurs with Mr. Bazalgette, who, in disclaiming any intention to take the water of excessive storms into the intercepting sewers of his London main drainage system, says, of a rain-fall of 2.64 inches in nineteen hours, "to intercept such a volume as resulted from this rain would have necessitated sewers of the capacity of rivers." Each of these engineers had a great problem to solve, from which the task of controlling storm water must be eliminated. Mr. Bazalgette passed it through his intercepting sewers, which carried all of the sewage of the metropolis, by overflows, into the old va'ley lines of sewers. Mr. Rawlinson, having for his object the rapid extension of sewerage facilities to the numerous village cities of Great Britain, the cost being the paramount consideration, advocated the minimum outlay for the smallest possible sewers, and advises carrying off flood water "over the surface," and by "existing road drains and drains to natural streams in valley lines," such as were generally to be found in these localities.

Mr. J. Bailey Denton, having in view the ultimate disposal of sewage, writes:

"To ascertain the amount of surface water to be admitted into sewers, or to be discharged by separate surface drains, it is necessary to have regard to the rain-fall of the district, and the maximum downfalls to which it is subject."

* * * * * * *

"It is in fact with considerable difficulty that the engineer arrives at the amount of surface water for which he must provide, in one way or another, by admission into the sewers, or by discharge through surface drains."

Mr. Baldwin Latham, the advocate of the combined system, and perhaps the highest authority upon this subject, says of deep sewer drains for the common conveyance of rain-water and sewage:

"Having ascertained the amount of rain falling in the district to be dealt with, which, for the purpose of being practically useful, should show the maximum quantity of rain that falls in the shortest time, all sewerage works must be calculated to convey away, either by the sewers

or by storm-water overflows, or special works constructed for the purpose, the maximum amount of rain without flooding or inconvenience to the inhabitants of the district."

This opinion would apply with equal force to the so-called clean-water sewers, which have been proposed to take the rain-fall in the separate system. The question cannot affect the relative cost of the two systems. Given the same duty to carry storm water, and the same capacity of conduit for this purpose is required in either case. The difference in cost will be always that due to the difference in depth of excavation; and against this the combined system, with its single deep sewer-drain, urges the whole cost of the additional small sewer required by the separate system. The engineers quoted are recognized leaders of the profession in this specialty; they agree in recognizing the difficulty of dealing with excessive storms, and while prescribing different methods of making provision for them, it occurs to neither of them to advise that such storms be left to take care of themselves because of their infrequency.

In 1878, the Common Council of Brooklyn resolved that "Whereas the matter of constructing storm sewers is a subject of very great importance to several localities of the city, which sadly need relief from the effects of sudden falls of rain" * * * "competent consulting engineers" should be selected, "and a full report be made by said engineers at an early day, giving their opinion as to the best and most economical plan for permanent relief."

These engineers reported as follows: "Your engineer has for several years been aware of the importance of improving the sewerage system, and the frequent complaints of householders in certain localities of the city have caused the most careful investigation to be made from time to time. The flooding of basements and cellars depreciates the value of property and endangers the lives of those occupying the flooded dwellings. * * * We find an unfortunate basin. * * * We find streets a mile or more from the water front graded about 8 feet above high-water level, and entire districts but a little over 10 feet above high tide. * * * We are convinced that many of the main sewers have become too small since the districts have been built over, and are, in many instances fixed at too low a grade. The lower portions of many districts are frequently inundated. * * * Grades have been established which make pockets or basins, and all the rain-fall not absorbed or evaporated must be discharged through the sewers."

They note the actual rain-fall of the locality, including precipitations of 1.65 inches in one hour, 1.2 inches in 40 minutes, and 2.6 inches in 25 minutes, and remarking that "In this view, and with the knowledge obtained from rain records, we have thought best to discard entirely English formula and try something applicable to our situation and locality; they recommend the construction of a number of relieving sewers, of large capacity at a cost of $750 000."

Practical Considerations.—When a drainage basin which includes a large city extends far beyond the probable extension of the latter; the

stream flowing through the city will ordinarily have a natural channel, which will command respectful consideration in time of flood. It is generally the lesser streams of smaller basins lying mainly within the limits of the present and prospective city that become the objects of empirical formulas for replacing the natural channels with artificial drains. In large cities and their suburbs, over which the conditions of a dense population will sooner or later extend, the question of disposal of storm water is not affected by the present condition of such suburb; the engineer must provide for the future. In all cities rain falling upon the surface of the streets and open spaces finds entrance to the drains only by street inlets, and may be excluded from them; falling upon roofs above the level of the streets it may be turned upon the streets; falling upon back yards, or upon areas below the street grade it must enter the drains unless surface drainage be provided by alleys in the rear. The proportion of rain-fall which may flow upon the streets will consequently vary from the whole amount to perhaps 60 per cent., the latter including only rain falling upon streets and roofs. It follows that 40 per cent. of the rain-fall is ordinarily the greatest proportion that must flow directly to the drains. The 60 per cent. falling or turned upon the streets may be left there, or any desired proportion may be turned into the drains by the street inlets. Whether it be advisable to make use of the street surface as a channel for storm water—for all, or for any part of it—will depend upon the local conditions; but, ordinarily, the inconvenience of conveying storm water upon the surface constrains all large cities to desire its conveyance under ground. Street surfaces are poor substitutes for natural channels.

It will often happen that long valleys of slight inclination, and even large basins without surface outlets, compel the underground conveyance, under penalty of inundation. Frequently the surfaces of streets covering a large tract of land are placed so far above the general elevation that to fill up the lots to street grades for surface drainage would cost very much more than would the additional capacity of main drains required to take from such areas the total outflow of excessive storms. It will sometimes happen that a system of storm drains which has proved too small may be relieved by conveying excessive storm water over streets to relieving or intercepting drains; and at all times existing works for the conveyance of storm water may be turned to the best possible advantage.

The use of the main sewers as subways for water and gas pipes and other municipal service, as in Paris, requires ordinarily a capacity so great that the question of capacity for rain-fall is eliminated, leaving only the necessity for adapting the shape of the invert to the dry weather flow. In the same city the conditions will vary in different drainage areas, or sub-areas, but there are only two general cases in which provision must be made for excessive storms; one, in which the drains may be relieved of some proportion of flood water by carrying it off upon the surface; and the other, in which the drains, including existing works, must carry it all. If storm water should gorge the main drains, their backwater will flood cellars and basements connected with them. Automatic or positive action flood-valves will give a precarious protection in certain cases of back flow from the main drain, but fail too often, and prevent all outflow while they operate to exclude the flood. The cost of such a valve is ordinarily many times the cost of giving the additional capacity required for excessive storms to that portion of the main drain in front of the premises, and is imposed upon the sufferer alone. The value of protection from such flooding with foul water can only be estimated by those who have experienced such flooding, and have noted the depreciation of their property which has resulted from this risk imposed upon it. It is doubtless preferable in such cases to avoid any openings from the premises into its house drain below the hydraulic grade line of the gorged sewer, but where the basements are occupied as kitchens, etc., or the premises depend upon the drain for the removal of ground water, this is not practicable.

Object of Drainage Works.—Drainage works have in every case a specific duty to perform—the conveyance of the maximum flood-flow from the proportion of rain-fall assigned to them, "without flooding or inconvenience to the inhabitants of the district." This duty is equally imperative, whether circumstances permit the exclusion of a large proportion of rain-fall from the drains or compel them to take it all. If the surface flow upon the streets passes off to a proper outlet without causing damage or inconvenience, the flood is well disposed of. If not, there is danger in permitting storm water to accumulate upon streets with steep grades. It becomes a torrent flowing with great velocity, and cannot then be captured by inlets designed to arrest, each, its share of shallow gutter flow with small velocity. It moves rapidly down to valleys, or basins without surface outlet; here it floods the surface because inlets

to receive it as fast as it comes can rarely be constructed—even should the drains here be of sufficient capacity. Inlets for large volumes of water in city streets are apt to be pitfalls for pedestrians and traps for cart wheels and horses' feet. If the drains of the inundated district are of insufficient capacity the consequences are, of course, disastrous. When an intercepting drain is designed to relieve such a district a definite work is assigned to it—the carrying off of all flood water falling upon the district above it, and there should be no danger of failure. Any intercepting drain should be of unquestionable capacity, because adjoining it are the smallest mains of the next sub-area, and a down-pour taxing the intercepting drain to its full capacity would rarely respect the boundry of the area, but would tax equally these small mains of the lower system. In this condition they are not fitted to receive any rejected surplus from the intercepting drain, which would flow to them over the surface. When drains are designed for basins without surface outlet, or to relieve such basins, they must of course, carry all flood water from the area which drains to them. Whether the drains must carry all, or part of the outflow from excessive storms, and whatever may be the area covered by down-pours or their duration, each sub-area of a size to be affected by it must have suitable provision in the capacity of its drains for such excessive down-pour. This compels the greatest capacity for all of the lesser mains and their lateral branches, which correspond to the upper sub-area in each drainage area, and together constitute a great part of a system of drainage. The debatable ground is, therefore, the principal main drain, and in deciding upon this in the presence of conflicting opinions as to what is required, the element of cost is too often decisive. It is proposed to show the actual relative insignificance of comparative cost.

Every main drain is like a bridge that carries over a stream the concentrated travel of many roads, a necessity to which all this travel pays toll. When a drainage area extends beyond city limits and beyond their probable extension, the cost of works which benefit only city property should not be imposed upon land in that area which will not share the benefit; but within these limits the cost of a main drain is properly chargeable to each beneficiary in proportion to the service rendered. Each square foot of area drained by it imposes an equal duty; derives an equal benefit. The cost should be equally distributed upon each square foot of area contributing its waters to the drain. This principle

has been generally recognized, but it has sometimes happened that the cost of a main drain has been otherwise provided for. When the general taxes of a city are applied to this purpose, outlying drainage areas do not get their drains until long after their proprietors have commenced to contribute towards the cost of constructing drains in other areas. If all taxpayers eventually receive the same consideration, this may be looked upon as a small loan without interest while the lenders are waiting, but the distribution of cost is not the same as in the drainage area tax for this reason: general taxes are assessed in proportion to value of property, both real and personal, areas entering only as one factor. The result of paying the cost of drains from general taxes is that the beneficiaries pay in proportion to their means, and not in proportion to the service rendered. In general it will happen that those who pay the greater portion in this way are the first to derive substantial benefit, owning the most valuable lands and being the first to feel the need of the drain; and this may be held to offset the greater burden imposed.

The special assessment plan imposes a certain proportion of the cost upon property abutting on the line of drain, upon the theory of immediate benefit. This need not prevent the equal ultimate distribution of cost. When an intercepting drain accomplishes the diversion of upland water from low grounds which cannot receive it without injury, the benefit of the drain extends far beyond the drainage area which it serves; but if the benefit be merely incidental to the service of its own area, it is, perhaps, doubtful whether any proportion of the cost should attach to the benefit on the low grounds. Within the limits of city areas and their probable extension, the cost divided by the drainage area, or the cost per square foot of drainage area, is the unit of value of service rendered. Whenever it is not necessary to modify the rule, each beneficiary will pay in proportion to the area of his land. From the record of about ten years sewer building in Washington, D. C., tables of cost, etc., have been prepared. (See Appendix.) From these the following statement has been deduced, assuming the area to be drained by a 12-inch pipe as the unit of area, and the cost per lineal foot of 12-inch pipe as unity. It is an approximate generalization.

RELATIVE COST AND CAPACITY of main drains required for various areas (without regard to slope or other characteristic of water-shed, assuming only uniformity of such conditions in the areas compared).

Col. 1.	Col. 2.	Col. 3.	Col. 4.	Col. 5.	Col. 6.	Col. 1.	Col. 2.	Col. 3.	Col. 4.	Col. 5.	Col. 6.
1	1.000	1.00000	.39200	.60800	$0.01303	80	5.210	.06500	.03784	.02716	$0.00058
2	1.174	.58700	.28100	.30600	.00656	90	5.500	.06100	.03531	.02569	.00055
3	1.388	.46200	.23160	.23040	.00494	100	5.730	.05730	.03325	.02405	.00052
4	1.567	.39200	.18190	.21010	.00450	200	8.640	.04320	.02407	.01913	.00041
5	1.743	.34800	.15800	.19000	.00407	300	11.730	.03910	.02088	.01822	.00039
6	1.920	.32000	.13700	.18300	.00392	400	13.300	.03325	.01917	.01408	.00030
7	2.090	.29900	.12180	.17720	.00380	500	14.790	.02958	.01815	.01143	.00024
8	2.250	.28100	.11090	.17010	.00365	600	16.280	.02713	.01746	.00967	.00021
9	2.410	.26800	.10530	.16270	.00349	700	17.770	.02539	.01697	.00842	.00018
10	2.570	.25700	.10000	.15700	.00336	800	19.260	.02407	.01583	.00824	.00018
20	3.170	.15800	.06500	.09300	.00199	900	20.750	.02305	.01411	.00894	.00019
30	3.480	.11600	.05040	.06560	.00141	1 000	22.240	.02224	.01274	.00950	.00020
40	4.030	.10000	.04330	.05670	.00122	2 000	36.300	.01815			
50	4.170	.08300	.04320	.03980	.00085	3 000	50.340	.01678			
60	4.610	.07700	.04190	.03510	.00075	4 000	50.970	.01274			
70	4.910	.07000	.03992	.03008	.00064						

NOTE.—For the purpose of comparison the diminution of proportional outflow with increase of area is neglected in the table, the increase of capacity required being assumed directly proportional to the increase of area. This is incorrect, but serves the purpose of making such comparison more unfavorable to the increase of capacity, and obviates the necessity for assuming any doubtful formulas.

COLUMN 1. Units of capacity or units of area drained, the unit being the capacity of a 12-inch pipe for conveying water, or the area which it will drain.

COLUMN 2. Relative cost per lineal foot of main drain for this area, the cost for the unit of area being assumed as unity.

COLUMN 3. Relative cost per lineal foot per unit of area drained.

COLUMN 4. Relative cost of a quadruple capacity per lineal foot per unit of area drained.

COLUMN 5. Excess of cost of a quadruple capacity per lineal foot per unit of area drained.

COLUMN 6. Assuming the unit of area at 6 acres, the unit of cost at $1 40 per lineal foot, and the average area of the ordinary city lot, including its proportion of street surface, at 4 000 square feet, the excess of cost to each lot in the drainage area, for quadrupling the capacity of its principal main drain, will be per lineal foot of drain, as shown in this column.

It will be seen that there is a great discrepancy between the increase of cost and the increase of capacity, nearly four thousand times the capacity being obtained with fifty-one times the cost; and that the excess of cost per unit of area drained to obtain a quadruple capacity diminishes rapidly as the capacity of the drain increases. Since the question of cost will arise upon the large main drains, the expediency of adequate provision for excessive storms in the smaller mains and laterals being admitted,

the decision to be made is, where shall the limit be placed? to what area shall we give this insurance against excessive storms? Being in doubt, the fact presents itself that the higher we place the limit, the easier rests the burden of insurance upon all whose lands, being themselves secure, send down the dangerous floods to the point in doubt. It it be in question to give this insurance to premises where the abutting drain conveys the water from 500 units of area or 3 000 acres, the pessimist who has been flooded by a gorged sewer may point out to the optimist who has not, that the cost to those already insured of extending a like protection to others cannot exceed one-fourth of 1 mill per lineal foot of drain for each average city lot. It is true that the sufferers from a gorged drain are only those whose property abuts upon the line of it, and beginning with the small branches at the summit, if all are made of a sufficient capacity down to some large main drain, those above the influence of the back water of the latter are apt to say, "After us the deluge;" but this is not equity. The same considerations which grant immunity to property bordering upon the lesser mains must apply with equal force to property lower down, otherwise the system of mains will not render equal service to all. The cost of this insurance, or the price of a factor of safety, is most equitably chargeable *pro rata* against those whose drains send down the flood with safety to themselves. Considerations of temporary expediency, which are always on hand to emasculate every project, should not be permitted to influence the proper expenditure. A factor of safety is an indispensable element of all engineering structures. It has been called the "factor of ignorance," because it makes allowance for what the engineer does not know. If the factor were proportional to the extent of this ignorance it would be very large in all hydraulic constructions, including sewers and drains.

Application to Washington, D. C.—While the writer has no theory to advance, no advice to give and no formula to offer, it is thought that the application made of the foregoing to the District of Columbia may be of interest to the profession. In 1871 the District of Columbia was first organized by Congress upon a basis admitting of a comprehensive plan of public work. From this dates the inception of plans for sewerage, the combined system being adopted. In 1874, a change in the form of government found the work well advanced, and many miles of brick and pipe sewers laid, ranging from 12 inches to 30 feet in diameter or span. The new administration, after four months' service, made their

first annual report, approving, generally, the work of their predecessors. The sewer system had been planned in harmony with views, which were then quite generally entertained, as to capacity required for the conveyance of storm-water. During the following year the new administration learned its own mistake and that of its predecessors; and being convinced alike with the property owners whose premises were flooded, that excessive storms should be provided for in a system of sewers, in their report for 1875 they predicted the destruction of the largest main sewer in Washington because of its insufficient capacity, and urged the construction of relieving sewers. Relief was attempted by reducing the size of the connecting pipe from the street inlets, and so excluding from the drains nearly all storm water which flowed upon the street surface. This sent down the floods to deluge the low-grounds, and was of little use at the higher elevations because of the large proportion of rain-fall entering the drains from roofs and back-yards, and from surfaces below grade. In 1876–77, the peremptory curtailment of expenditures by act of Congress put a stop to all attempts at relief, but in the report for 1876 the warning is renewed. In 1878, 500 feet of the principal main sewer of Washington was destroyed by flood-water, as predicted, and the complaints of sufferers from back-water of gorged sewers were universal. The new form of government of the District of Columbia was made permanent, and the attention of Congress having been secured, plans were presented for the present system of relieving sewers upon which work was commenced in 1879.

Topography.—The topographical sketch, Plate XII, gives the established grades within the city limits, and the natural surface outside these limits, covering all territory draining through the City of Washington. There are three principal depressions within the city, shown by the shaded surface on the map. One of these, an elliptical basin without surface outlet, has its longer axis on L street, N. W., between Vermont avenue and 23d street, a length of about $\frac{9}{10}$ of a mile, and has a mean breadth of about $\frac{1}{4}$ of a mile. The rim of this basin has an elevation of 60 feet above tide, and the lowest point within it is 12.3 feet below the rim. The steep slopes on the north and south of the basin send down the rain-fall rapidly towards it; and while existing sewers were gorged, the water accumulated to the depth of $4\frac{1}{2}$ feet upon the surface. Between this basin and Boundary street on the north, were, originally, the slashes and the deep valley of Slash Run. These have been filled by intersecting streets to a depth varying from 4 to 40 feet, leaving numerous

squares without surface outlet, and so far below street grades that owners opposed, with violence, any proposition to bring them to grade. This is known as the Slash Run District. Another depression extends from 17th street west to the foot of Capitol Hill, the rim of the basin being 10 feet above tide, and the lowest point (about at the middle of this area) 3.4 feet below the rim. It is about $1\frac{4}{10}$ miles long and 1 000 feet wide. It is separated by a low divide from another depression, extending southward about $1\frac{4}{10}$ miles to the Eastern Branch and tapped by the James Creek Canal.

Through these low grounds the two principal main sewers of Washington discharge into the Potomac, receiving from them numerous tributary drains, which are quickly affected by back water in the main sewers. It will be seen that the principal thoroughfare of Washington (Pennsylvania Avenue) follows the northern boundary of this area, which occupies the central part of Washington City. The ground falls precipitously from the summit, 320 feet above tide, to the city limits, and quite rapidly through the city to the low grounds. In descending to the boundary of this low area, only two-thirds of the distance to the Potomac is accomplished, and 310 feet of head are lost, leaving only 10 feet with which to accomplish the remaining third of this distance. The natural surface of Washington and vicinity is generally a stiff retentive clay, which is almost impervious to water. In laying pavements, "wet holes" in this material have to be dug out and refilled with dry earth, because water would never escape from them. The street pavements are of stone blocks with tarred joints, coal-tar concretes, and asphalt; the latter are preferred wherever it can be advantageously put down, and all are practically water-proof.

Meteorology.—The records of the Smithsonian Institute, of the Naval Observatory and of the Signal Service, are available for the determination of rain-fall. The latter contains the continuous curve record of a self-registering rain-gauge from 1871 to the present time, giving the intensity and duration of down-pours with the detailed phenomena of all storms. From these sources and principally from the self-registering rain-gauge of the Signal Service, it is found that the following rain-falls are to be expected in the District of Columbia :

Table of actual intensity and duration of rain-fall in the District of Columbia:

Duration in hours.....	0.083	0.133	0.250	0.333	0.366	0.416	0.500	0.666	0.750	1.25	3.25	56.00
Rate in inches, per hour...	6.24	3.53	3.00	2.22	2.65	4.27	2.28	3.00	1.52	1.00	1.51	0.14

In Philadelphia, Pa., a rate of 3 inches per hour has continued two hours; in Norristown, Pa., a rate of 1.8 inches per hour has continued five hours. The proximity of these localities to the District of Columbia and the similarity of meteorological conditions, suggests that these observations be included with those in the District of Columbia, which cover only a very limited period of time. These observations are all plotted on Plate XIII.

This record being mainly that of a single rain-gauge, the area covered by the several precipitations cannot be stated. It is inferred from the effect, at times, upon the several drainage areas that they may cover from 5 to 50 square miles, if not more. The principal freshets of the Potomac occur from February to April. Violent rain storms in the District of Columbia occur, usually, between July and October, both months inclusive, and they are more apt to occur in one or the other of these months, rarely much earlier than July or later than October. Freshet and local storm will, therefore, rarely coincide. Violent precipitation will seldom occur upon frozen or icy surfaces, but immunity from this cannot be depended upon; and this is not important, as the growth of the city will cover all natural surface which drains through it now.

Proportional Outflow of Storm Water.—In the absence of exact determination of areas covered by individual storms, the total precipitation upon any drainage area cannot be ascertained and the precise relation between rain-fall and outflow cannot be traced. It has been found, however, that when the gauge at the Signal Office registered a rainfall of 1.78 inches in twenty-five minutes, an area of 436 acres, in which the Signal Office occupied a central position, and which is mainly paved and built up, gave a maximum rate of outflow between 2 and 4 inches per hour; the main drain with a capacity of 2 inches per hour, running under a high head, and a large quantity of storm water escaping over the surface. The gauging of the main drain of the same area in 1885, at a point where it receives the drainage of 200 acres, paved with asphalt and well built up, indicated no appreciable loss during the period of maximum flow when the rain-gauge gave a precipitation of 0.55 of an inch in thirty-seven minutes, the rate of outflow being exactly the rate of rain-fall. When the rain-gauge gave a precipitation of ½ inch in fifteen minutes, a rate of 2 inches per hour, the maximum rate of outflow in the drain was three-quarters of the rate of rain-fall, or 1½ inches per hour. Gauging the original main sewers

was, generally, of little use, because when not gorged the rainfall was insignificant, and when gorged by heavy rains the proportion escaping over the surface could not be measured. Tiber sewer was destroyed when discharging about 3 000 cubic feet per second, but the record is complicated with pondage, overflow, and partial diversion of the storm water. Upon another occasion the water-ram in the easterly main branch of this sewer, 9½ feet in diameter, ruptured the branch sewer, throwing a fragment of the arch weighing several hundred pounds over a distance of 50 feet.

Plans.—On the topographical sketch, Plate XII, are shown the three principal original main sewers. To relieve these the principle of interception was adopted; the relieving sewers also shown being designed to carry, without risk of failure, all surface water flowing from the respective areas above them. To harmonize with the existing system they were designed as sewer drains, having generally an oval shape, excepting that section of the intercepting storm sewer on Boundary street, between the point *A* and the Eastern Branch. It was decided that sewage should not be taken toward the Eastern Branch because of its unimproved channel and of the wide marshes on either side which might wait long before reclamation. For this reason the whole line from 14th street to the Eastern Branch was at first designed as a storm conduit or overflow sewer. The plan was subsequently modified to make this line a sewer-drain as far as point *A*.

The computation of capacity for the relieving sewers and their lateral branches assumed a maximum rain-fall of 3 inches per hour (the rate which ruptured the old Tiber sewer) and a maximum outflow of two-thirds of this, or 2 inches per hour. It was known that with this assumption the smaller sizes of main sewer must run, at times, under a head, being then subjected to an internal pressure, which they were fitted to sustain; but it was thought this would never occur where the sewers were of a size to make such pressure dangerous—the factor of safety increasing with the size of the sewer, or with the quantity of water carried, and therefore with the relative importance of this consideration. These sewers having to supplement to the best advantage an existing system, required, each, a special study. The general considerations have been given at length hereinbefore. The details of all would be tedious, and the largest of these, the main sewer in Boundary street, between 14th street west and the Eastern Branch, will serve as an illustration.

Boundary Intercepting Storm Sewer.—This was designed with a

minor drain on Maryland avenue for the relief of Tiber sewer, to take from it all of its drainage area except about 1 500 acres. The diversion of the drainage of this area leaves the Tiber sewer subject to a second catastrophe, by the same conditions which caused the first one, if it should ever be required to discharge from its remaining drainage area the maximum flow assumed of 2 inches per hour, or two-thirds of the rate of rainfall which ruptured it (3 inches per hour). The line of Boundary street crosses the outlet valley of the precipitous area to the north of the city, with a high embankment. On this line is constructed the boundary intercepting sewer.

A glance at the map shows that Washington is hemmed in on the south by the Potomac, on the east by the marshes of the Eastern Branch, and on the west by the deep gorge of Rock Creek Valley. It can expand only northward over the area drained by the boundary intercepting sewer. Soldiers' Home Park, Glenwood Cemetery and the new reservoir occupy about 660 acres of this, upon the highest ground. The precipitous slopes of the park can give no pondage, and its stiff soil but little absorption. The well-kept walks and drives permit the rapid discharge of storm water. The park may resist the encroachment of the city, or it may melt away in time as other military reservations have done under similar circumstances. The cemetery, of course, must go sooner or later. Here are found the finest building sites in the District of Columbia. Over the greater part of this entire area, which is already to a great extent subdivided and occupied by suburban residences, the impervious roofs and pavements of Washington will eventually extend. Having steeper slopes, there will be a more rapid discharge of storm water than obtains anywhere within the present city limits. In addition to this suburban property it was necessary to divert into Boundary sewer the drainage of a large city area between 14th, O and Boundary streets by a main sewer along O street. It will be noticed that the O street sewer, the main artery of this area, crosses a deep depression, the former valley of Tiber Creek, to reach the Boundary sewer. To prevent back flow in the former, it was necessary to keep the level of storm flow in the latter as low as possible. Here came a serious difficulty. The great depth of excavation and the character of the ground which former excavation had shown would be encountered, together with the necessarily great size of the Boundary sewer, developed

a complicated question—the proper form and size to be given to this great main drain to satisfy all existing conditions.

At the point *A* this question was presented, where a concentration of flow occurs, which continues without material change to *D*. The rapid convergence to this point of the drainage lines of its four principal sub-areas is indicated by the sketch. The surface of the first sub-area falls from the summit, 320 feet above tide, to 60 feet above tide at *A* in a distance of about 13 200 feet, having a general slope of 19.69 feet per thousand, and draining about 1 016 acres. The surface of the second sub-area falls from the same summit to 70 feet above tide at *B* in a distance of 11 800 feet, having a general slope of 21.18 feet per thousand, and draining about 915 acres; from this point it continues to *A* through Boundary sewer. The surface of the third sub-area falls from 220 feet above tide to 88.5 feet above tide at *C* in a distance of 6 300 feet, having a general slope of 20.87 feet per thousand, and draining about 320 acres; from this point it continues to *A* through Boundary sewer. The fourth sub-area is that within the city limits; the surface falls from 127 feet above tide at 14th and Boundary, to 60 feet above tide at the point *A*, in a distance of 9 100 feet, having a general slope of 7.36 feet per thousand, and an area of 485 acres. In each case the natural drainage lines are but little longer than the air line distances given above. The aggregate of these four sub-areas gives an extent of water-shed tributary to Boundary sewer at the point *A* of rather less than $4\frac{1}{3}$ square miles. This is less than the probable area covered by the down-pour of June 28th, 1881, giving a rate of precipitation of 4.23 inches per hour; it is much less than the probable area covered by the down-pour of August 5th, 1878, giving a rate of precipitation of 3 inches per hour for forty minutes. The surface slope in feet per thousand in these several sub-areas ranges from 0.92 to 2.64 times the slope in the 200-acre paved area of New York avenue sewer, which gave by gauging a maximum rate of outflow equal to the maximum rate of precipitation. The character of the surface of these sub-areas will eventually be nearly the same as that of the New York avenue area, and must sooner or later combine with their steeper slopes to produce the same results.

A mean velocity of 6 feet per second would bring to *A* the rain-fall upon the point most remote from *A* in forty minutes. In that time a storm covering the $4\frac{1}{3}$ square miles of territory in the four sub-areas

draining to A would therefore develop its maximum effect at A; the maximum flood wave at A receiving then, a simultaneous contribution from every point within this water-shed. It was a rain-fall of 2 inches in forty minutes, a rate of 3 inches per hour, that destroyed the Tiber sewer, the relief of which is sought by the construction of the Boundary sewer. It was evident that the maximum capacity here must not be less than elsewhere in the new system. This fixed the maximum capacity at two-thirds the maximum rate of rain-fall assumed (3 inches per hour); and the existing conditions of slope, etc., gave by Kutter's formula a conduit of a circular cross-section and 20 feet diameter, for conveying this maximum flow.

All of the foregoing applies with even greater force to the smaller sections of Boundary sewer above the point A. At this point other important considerations present themselves. This section of Boundary sewer, at the grade adopted, requires an average depth of excavation of 45 feet, reaching at one point, near the point A, a maximum depth of 50 feet. Had it been assumed that the full capacity would be required except at long intervals for phenomenal storms, the sewer must have been laid about 10 feet deeper at A, with increased section to compensate for the diminished slope, in order to keep the surface of flood-flow below the danger line for the O street sewer. The depth was fixed by the assumption that the ordinary and even extraordinary maximum flood-flow would be one-third of the maximum rate of rain-fall, and would be carried in a semicircular conduit, the flood surface being at the spring line of the arch. The semicircular section gives the maximum sectional area adapted to variable flow, with the minimum height of flood surface, for ordinary storms. A factor of safety of two is given by the additional space between the spring line and the soffit of the covering arch, but does not cover the safety of the O street area. This must be jeopardized by any lifting of the ordinary flood surface above the level assigned to it. At the grade adopted, this section of Boundary sewer is constructed, as foreseen, throughout nearly its entire length in a stratum of fine white sand, which becomes quick sand when complicated with pockets of water, but is firm enough when undisturbed. The thrust is taken, generally, by this material, which would be a treacherous backing under frequently varying conditions of saturation of the sand and internal stress in the sewer. The construction is indicated in the diagram. See Plate XIV.

Such a large masonry conduit, in such a formation, at great depth, depends for its safety upon being undisturbed, and this assurance it is intended to give with the factor of safety adopted. To illustrate: suppose that the maximum capacity were reduced to, say, six-tenths of that assumed, the conduit receiving a semicircular invert of 15 feet diameter and an interior height of 17½ feet. Such a conduit would be taxed to its utmost capacity by a maximum outflow about one-fifth greater than would be carried in the invert of the larger conduit. With a greater outflow the smaller conduit must run under a head, becoming a forced conduit.

Conduits running full-bore are subjected to an outward pressure, due to the contained water, which increases as the vertical diameter and the perimeter of the conduit. The dangerous feature of this hydrostatic rupturing pressure is not, ordinarily, the lifting effect upon the arch, but the horizontal thrust which induces a spreading at the springing line and a depression at the key. It is by such deformation that masonry conduits fail when the arch is heavily loaded and the thrust passes out into a yielding ground. The tendency of this hydrostatic pressure is to reinforce the thrust of the arch. Under the conditions stated, this conduit is subjected to dangerous strains whenever the flood surface rises above the springing line of the arch. If it run only full bore it will sustain a hydrostatic pressure of about 14 tons per lineal foot; or over a section of 20 feet, behind which a cavity might form, a fluctuating pressure reaching 280 tons. If run under a head, the pressure is greater, and the fluctuating storm wave will develop the much more dangerous water-ram. A too frequent hammering of this kind is a matter for grave consideration in such a structure at great depth in a sand formation, and more especially if a mistaken economy be permitted to reduce the thickness of arch or invert. A conduit designed to run full-bore in ordinary storms is habitually exposed to both these sources of danger, whereas another of such capacity that the fluctuations of surface flow in ordinary storms are kept within the limits of the invert, will incur less danger in phenomenal storms at long intervals. It is perhaps unnecessary to show that such a diminution in size of conduit would defeat the principal object secured by the larger conduit—the lowering of the flood surface and the conveyance of ordinary storms in the channel of maximum sectional area adapted to variable flow, with minimum height of flood sur-

face. A storm that would carry the flood surface in the larger conduit to the danger line for the O street area, would carry it 6½ feet above that line in the lesser conduit.

The area drained by Boundary sewer at the point *A* and the area remaining to Tiber sewer, aggregate 6.7 square miles. A storm covering this combined area would tax simultaneously the capacity of each, and if Boundary sewer fail, Tiber sewer must take the consequences. It has been already explained that disaster will follow if the rate of flow in Tiber sewer equal two-thirds the rate of rain-fall which ruptured it in 1878. The Nagpoor, India, storage reservoir (see *ante*) receives the flow from a water-shed of 6.6 square miles, an area almost identical with the combined area drained by Boundary and Tiber sewers. With a very absorbent natural surface, that water-shed has nevertheless delivered to the reservoir in one hundred and seventy minutes, 98 per cent. of a down-pour upon its entire area of $2\frac{2}{10}$ inches in eighty minutes, when the power of absorption of the soil had been satisfied. From this may be inferred the effect upon the area drained by Boundary and Tiber sewers, when the greater part of this area, including the steep slopes of the former, is covered by waterproof artificial surfaces, of a precipitation of nearly the same quantity of rain-fall (2 inches) in one-half the time (forty minutes) to which it has been subjected quite recently. It must be subjected to storms of even greater violence and duration. The impotency of empirical formulas to deal with this question may be seen in their application to the New York avenue intercepting sewer (see *ante*). The impropriety of petty economy in the design of this work, which closely affects the interests of nearly 7 square miles of valuable property, appears quite clearly in the record of the work it is designed to supplement.*

On the 5th of August, 1878, a rain-fall of 2 inches in forty minutes occurred in the District of Columbia, a rate of 3 inches per hour. It ruptured the largest main sewer of Washington—the old Tiber Arch. For a distance of 500 feet the brick arch, 18 inches in thickness, and having a span of 30 feet, was lifted bodily and broken at the haunches and at the crown. Fortunately it fell with the subsiding water into place again, the fractured edges projecting half way into the sewer. The sectional area of Tiber sewer is here 220 square feet. If the arch

* See Report of Commissioners, D. C., for 1878.

Details of the Sewers of Washington. See Diagram, Plate XIV.

No. of diagram. No. of reference	1 (1)	1 (2)	1 (3)	1 (4)	2 (5)	2 (6)	2 (7)	2 (8)	2 (9)
Interior dimensions	12″	15″	18″	24″	2′ x 3′	2′3″ x 3′4.5″	2′6″ x 3′9″	2′9″ x 4′1.5″	3′0″ x 4′6″
Relative capacity	1	1.84	3.05	6.68	10.81	15.17	19.4	25.51	32.46
Relative cost	1	1.15	1.40	2.04	2.70	3.00	3.15	3.33	3.57
Average depth of excavation	10′6′	10′6″	10′6″	10′6″	12′6″	12′9″	13′0″	13′3″	13′6″
Sectional area, square feet	.785	1.227	1.767	3.140	4.64	5.92	7.23	8.71	10.41
Perimeter, lineal feet	3.14	3.92	4.71	6.28	8.17	9.12	10.33	11.07	12.14
Hydraulic radius	.25	.31	.375	.50	.567	.649	.70	.787	.857
Diameter of corresponding circular sewer					2.27	2.60	2.80	3.15	3.43
Itemized quantities per lineal foot:									
Cubic feet, brick work					4.70	5.15	5.63	5.95	6.60
Cubic feet, concrete	1.57	1.96	2.32	3.07	4.45	4.60	4.75	5.00	5.24
Square feet, trap rock									
Diameter pipe invert					12″	15″	15″	15″	15′
Cubic yards excavation	1.00	1.10	1.20	1.40	1.80	2.00	2.10	2.30	2.40
Cubic yards surplus earth	.100	.13	.18	.27	.52	.59	.67	.74	.83
Itemized cost per lineal foot:									
Brick work					$1 39	$ 1 53	$1 67	$1 76	$1 95
Concrete	29	36	43	57	82	.85	88	92	97
Trap rock									
Pipe with connections and manholes	50	56	76	1 30	10	10	10	10	10
Pipe invert					25	33	33	33	33
Excavation and refilling	40	44	48	56	63	70	73	80	84
Surplus earth removed	02	02	03	04	07	09	10	12	13
*Total cost	$1 21	$1 38	$1 70	$2 47	$3 26	$3 60	$3 81	$4 03	$4 32
Increased cost of ramming earth	16	17	18	22	27	27	27	28	28
Aggregate cost	1 37	1 55	1 88	2 69	3 53	3 87	4 08	4 31	4 60

* In case of paved streets the cost of repaving is to be added.

DETAILS OF THE SEWERS OF WASHINGTON. SEE DIAGRAM.—(*Continued.*)

No. of Diagram. No. of Reference	2 (10)	2 (11)	2 (12)	2 (13)	3 (14)	3 (15)	3 (16)	3 (17)	3 (18)
Interior Dimensions	3'3"x4'10.5"	3'6"x5'3"	3'9"x5'7.5"	4'0"x6'0"	3'3"x4'10.5"	3'9"x5'7.5"	4'0"x6'0"	4'6"x6'9"	5'0"x7'6"
Relative capacity	40.15	49.29	60.42	70.69	40.13	59.05	70.13	95.75	126.44
Relative cost	3.89	4.14	4.40	4.71	4.19	4.80	5.16	5.67	6.15
Average depth of excavation	13'.09"	14'.00"	14'.03"	14'.06"	14'.03"	14'.09"	15'.00"	15'06"	16'.00"
Sectional area, square feet	12.13	14.18	16.50	18.63	12.12	16.16	18.38	23.26	28.71
Perimeter, lineal feet	12.95	13.95	15.07	16.10	12.88	14.86	15.85	17.84	19.82
Hydraulic radius	.937	1.01	1.09	1.15	.94	1.087	1.16	1.30	1.45
Diameter of corresponding circular sewer	3.75	4.04	4.36	4.60	3.76	4.348	4.64	5.20	5.80
ITEMIZED QUANTITIES PER LINEAL FOOT:									
Cubic feet, brick work	7.14	7.68	8.25	8.65	6.39	7.20	7.60	8.00	8.65
Cubic feet, concrete	5.41	5.81	6.06	6.45	4.21	5.300	6.00	6.70	7.33
Square feet, trap rock					3.38	3.73	4.00	4.50	5.31
Diameter pipe invert	18"	18"	18"	20"					
Cubic yards excavation	2.60	2.80	3.00	3.20	2.60	3.00	3.20	3.60	4.00
Cubic yards surplus earth	.92	1.00	1.15	1.26	.93	1.13	1.30	1.47	1.82
ITEMIZED COST PER LINEAL FOOT:									
Brick work	$2 11	$2 27	$2 44	$2 56	$1 89	$2 13	$2 25	$2 37	$2 56
Concrete	1 00	1 07	1 12	1 19	78	98	1 11	1 24	1 35
Trap rock					1 25	1 38	1 48	1 67	1 96
Pipe with connections and manholes	10	10	10	10	10	10	10	10	10
Pipe invert	44	44	44	55					
Excavation and refilling	91	98	1 05	1 12	91	1 05	1 12	1 26	1 20
Surplus earth removed	14	15	17	19	14	17	19	22	27
Total cost	$4 70	$5 01	$5 32	$5 71	$5 07	$5 81	$6 25	$6 86	$7 44
Increased cost of ramming earth	28	29	29	31	28	28	31	32	33
Aggregate cost	4 98	5 30	5 61	6 02	5 35	6 09	6 56	7 18	7 77

DETAILS OF THE SEWERS OF WASHINGTON. SEE DIAGRAM.—(*Continued.*)

No. of diagram. No. of reference	3 (19)	3 (20)	3 (21)	4 (22)	4 (23)	4 (24)	5 (25)	4 (26)	4 (27)
Interior dimensions	5'.06"x8'03"	6'.0"x9'0"	7'.0"x10'6"	7'6"	8'3"	13'2"	20'0"	20'0"	22'0"
Relative capacity	163.13	205.89	309.25	231.02	302.49	1013.82	3016.73	3016.73	3861.48
Relative cost	7.0	9.19	10.87	9.73	12.94	22.45	50.25	50.92	50.91
Average depth of excavation	16'6"	17'0"	18'0"	15'5"	15'9"	23'04"	45'02"	32'2"	23'2"
Sectional area, square feet	34.74	41.35	56.27	44.18	53.46	136.16	314.16	314.16	380.13
Perimeter, lineal feet	21.80	23.79	27.75	23.56	25.92	41.36	62.83	62.83	69.11
Hydraulic radius	1.59	1.738	2.027	1.875	2.062	3.290	5.00	5.00	5.50
Diameter of corresponding circular sewer	6.36	6.952	8.108						
ITEMIZED QUANTITIES, PER LINEAL FOOT:									
Cubic feet, brick work	10.43	15.00	17.16	12.68	18.40	37.92	28.21	95.17	104 49
Cubic feet, concrete	8.45	12.83	14.73	16.45	17.15	26.45	135.27	68 31	73.17
Square feet, trap rock	5.56	6.08	7.77	8.11	12.96	11.85	16.44	16.44	17.14
Diameter pipe invert									
Cubic yards, excavation	4.40	5.20	6.20	5.80	6.40	19.00	70.00	4.40	30.00
Cubic yards, surplus earth	2.16	2.75	3.45	2.94	3.62	7.75	16.20	16.20	18.90
ITEMIZED COST, PER LINEAL FOOT:									
Brick work	$3 09	$4 44	$5 08	$3 76	$5 45	$11 23	$8 36	$28 19	$30 96
Concrete	1 56	2 37	2 72	2 74	2 86	4 41	22 54	11 38	12 19
Trap rock	2 06	2 25	2 87	3 00	4 79	4 38	6 08	6 08	6 34
Pipe, with connections and manholes	10	10	10	10	10	28	40	33	28
Pipe invert									
Excavation and refilling	1 32	1 56	1 86	1 74	1 92	5 70	21 00	13 20	9 00
Surplus earth removed	32	41	52	43	54	1 17	2 43	2 43	2 83
*Total cost	$8 45	$11 13	$13 15	$11 77	$15 66	$27 17	$60 81	$61 61	$61 60
Increased cost of ramming earth	39	40	46	46	50	2 02	9 68	5 67	2 00
Aggregate cost	8 84	11 53	13 61	12 23	16 16	29 19	70 49	67 28	63 60

*In case of paved streets the cost of repaving is to be added.

The following prices have been assumed: Excavation and refilling, 40 cents per cubic yard from case (1) to (5); 35 cents per cubic yard from (5) to (18); 30 cents per cubic yard from (18) to (27). Hauling surplus earth, 15 cents per cubic yard Brick masonry, $8 per cubic yard. Concrete, $4 50 per cubic yard. Trap rock, 37 cents per square foot. In cases (24), (25), (26) and (27), excavation is measured allowing a slope of 1 foot horizontal to 2 feet vertical from springing line to surface, and average depths in (25), (26) and (27), as constructed.

had fallen in and obstructed the water-way, a torrent of 3 000 cubic feet per second would have been thrown upon Pennsylvania avenue. The cost of this work was about $108 per lineal foot. In this single instance $54 000 worth of mistaken economy was swept away in forty minutes. Tiber sewer had been but two years completed, but the cost of the section destroyed is insignificant when compared with the destruction and depreciation of property which this sewer had already effected.

DISCUSSION.

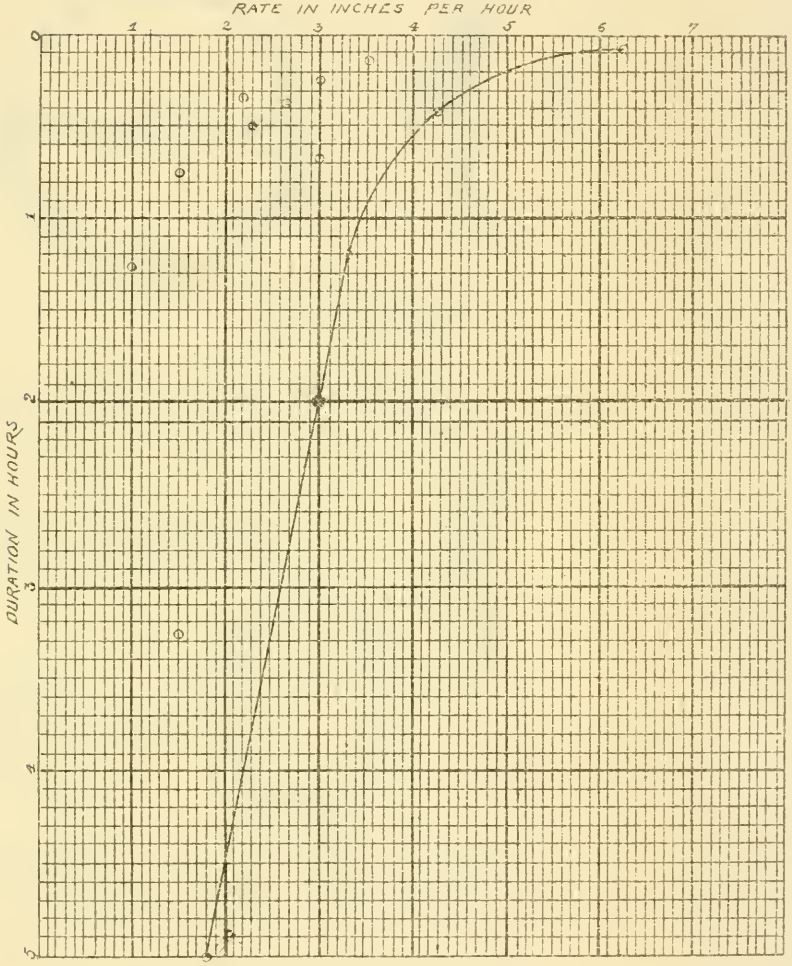
J. Foster Flagg, M. Am. Soc. C. E.—In a visit to the Caribbee Islands in the early part of 1885, the writer learned of a remarkable fall of rain that occurred in St. Kitts in January, 1880, and took some pains to investigate it and verify the accounts given of it. The rain-fall was of such a phenomenal character, so much exceeding any other that he had seen recorded, that it seemed to him worthy of record in the *Transactions* of the Society in connection with the valuable paper of Mr. Hoxie upon this subject. This outpour from the clouds, in which it would seem as if the water must have fallen by bucketfuls instead of in drops, partook, apparently, of the nature of the "cloud-bursts" of our western States, as they are there termed, but extended over a larger area than probably is common in the latter; it certainly was not a waterspout, as the excessive fall of rain was noted over an area of several square miles at least (a good authority estimates the area of intense rain-fall as 6 or 8 miles square). There was a light rain in the evening of the night on which the down-pour took place, but none of consequence before 10 or 11 P.M.; and before daybreak—by 5 A.M.—it was all over. There were no rain gauges—any ordinary ones in fact would have been entirely useless, but a tank with vertical sides, used in sugar making, at a sugar estate near the town of Basse Terre and the south coast of the island, which was known to have been empty before the rain took place, and which was 30 inches deep, was filled and running over; the tank was lying out of doors where it could have received no other water except that falling directly into it from the clouds. One intelligent informant said that at a low estimate 36 inches of rain must have fallen; the evidence is positive that over 30 inches fell, most of it in about six hours, and all in ten or eleven hours. At another sugar estate, on the opposite or northerly side of the island, 4 or 5 miles away in an air line, and with a mountain some 3 000 feet high intervening, a similar sugar tank (or copper), empty the night before, had 23 inches of rain-fall in it the following morning.

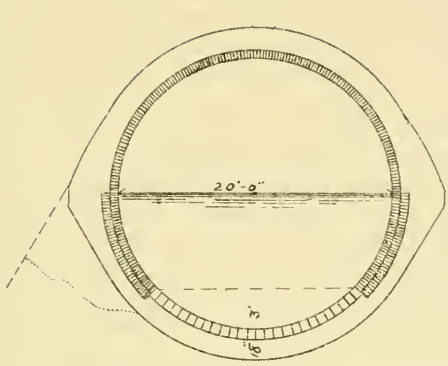
PLATE XII
TRANS. AM. SOC. CIV. ENGRS.
VOL XXV. No 489.
HOXIE ON
EXCESSIVE RAINFALLS.

TOPOGRAPHICAL SKETCH
OF
WASHINGTON, D.C AND
VICINITY.
SHOWING DRAINAGE LINES OF ALL TERRITORY
DRAINING THROUGH THE CITY.
1883.

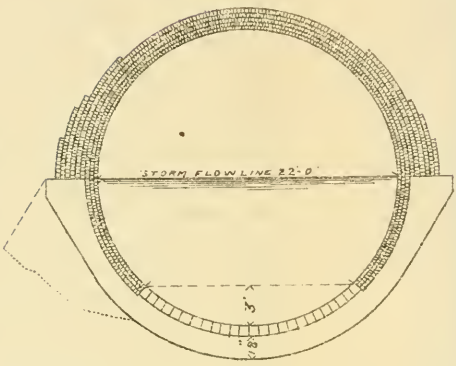
PLATE XIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL XXV. N° 489
HOXIE ON
EXCESSIVE RAINFALLS.

DIAGRAM OF MEAN INTENSITY OF HEAVIEST DOWN-POURS. PLOTTED FROM OBSERVATIONS IN THE DISTRICT OF COLUMBIA





5.



4.

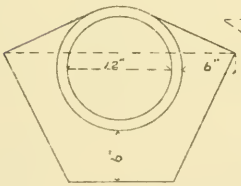
CIRCULAR SEWERS.

THE SEWERS OF WASHINGTON D.C.

CONSTRUCTED AND PROPOSED FROM 1874 TO 1883.

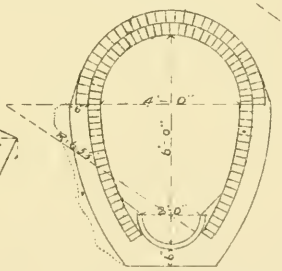
NOTE.

THE THEORETICAL FORM OF INVERT IS SHOWN IN PRACTICE THE CONCRETE IS RAMMED HARD AGAINST THE NATURAL EARTH SIDES OF THE EXCAVATION AS SHOWN BY DOTTED LINES.



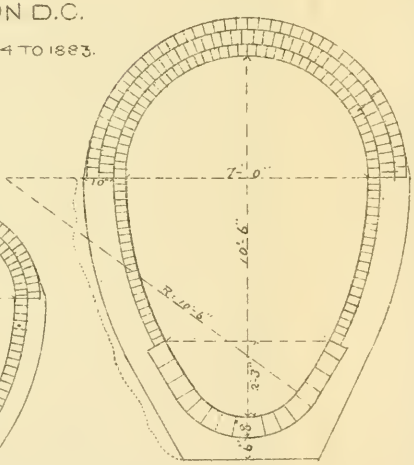
1.

TERRA COTTA PIPE SEWER.



2.

EGG SHAPED SEWERS, WITH HALF PIPE INVERTS,



3

WITH TRAP ROCK INVERTS.

The destructive effects of the storm upon the little town of Basse Terre, the capital of the island, are strongly corroborative of the direct evidence as to the magnitude of the rain-fall. The town is built on a plain along the shore, having a slight slope upwards away from the sea, and also in an easterly direction on the line of the shore. It has little depth from the sea in proportion to its length, and has a number of streets running down towards the shore; there is no semblance of a hill or valley on either side of the town. But in the rear of the town, a half or three quarters of a mile away, Monkey hill rises sharply up from the plain to an altitude of 1 150 feet—so sharply and with so little extent to its summit that the whole hill covers but little area on the plain. It is nearly isolated, connecting by a "hog back" with Mt. Olivees (over 3 000 feet high) a portion of the ridge which forms the backbone of the island. The whole island has an area of only 68 square miles, and but a small portion of it (perhaps 5 or 6 square miles) drains more or less in the direction of Basse Terre, though with no defined water-course. From the precipitous character of the mountainous water-shed, however, nearly the entire amount of the rain falling upon it that night must have been delivered rapidly upon the ill-fated town. Notwithstanding the shortness of the streets debouching upon the bay, and their number, the streets were filled with a torrent in places over 8 feet deep. Several of the smaller houses were swept down the streets and out to sea, and larger ones carried from their foundations into the streets. Over 200 persons were drowned, a large number being carried out to sea. Such a torrent naturally brought down with it a large amount of detritus; and in the public garden, a square of 450 feet each way, the northern half was filled with mud to a depth of 4 or 5 feet, entirely covering the fence surrounding it (the water was 6 to 8 feet deep in the square). In fact the following morning a woman was dug out of the mud who had been caught as she was attemping to cross the square, and buried upright as she stood, the top of her head alone showing. This square is only 225 feet from the open shore, with two streets respectively 30 and 33 feet wide leading from it directly to the beach. Two streets of similar width enter it from the land side, and two entering it (one at the the lower end) run parallel with the shore. If the town had been situated in a narrow valley one can readily understand how the water would have risen even to a much greater height; but with its numerous streets emptying directly into the open sea the height of the torrent therein was something marvelous, and indicates plainly the extraordinary nature of the rain-fall.

William Rumble, M. Am. Soc. C. E.—I find among my old papers a record of rain-fall at Morrisania, of which the following is a copy:

October 30th, 1866.

Rained most of previous night. From 7.45 A.M. until 9.15 A.M. the amount of rain-fall was 1.73 inches, being at the rate of 1.15 per hour.

At 10 A.M. there was running in Mill Brook at the railroad crossing,

near 3d street, 100 cubic feet per second, equals 360 000 cubic feet per hour.

The area of Mill Brook water-shed, above the point of measurement. is about 1 790 acres, and the amount of rain-fall on the same at the above rate will equal 7 472 355 cubic feet in an hour—showing the amount running in the stream to be about 4¾ per cent. of the rain-fall per hour.

Mr. R. E. McMath, M. Am. Soc. C. E.—I think that this Society ought to take steps to arrive at some definite information as to the necessary sizes for sewers, and that definite information I think could be obtained by getting together the experience which the different members of this Society have in their own possession. Now, in the City of St. Louis we have to do with this question of the maximum rain-fall, because the topography of our city is such that we have numerous depressed basins which have no natural outlet, and we must provide for the greatest quantity of water which will fall, and carry it off as rapidly as it reaches the sewers. We have those depressed basins ranging from fifty acres up to one thousand. We have one containing six hundred acres. We had a rain-fall on the 14th of May last of $3\frac{3}{10}$ inches, falling wholly within two hours, the greater part of it within forty minutes. By tabulating the sizes of the sewers, and their gradients on the areas which are graded at different parts of the length, and by considering the fact that at certain portions of that length the sewers are overcharged, and at others there is no difficulty experienced, we have made a determination of the size and the gradient which will carry off the greatest rain-fall which occurs in that locality. This method was adopted in St. Louis in determining the size of sewers nine years ago. No sewer that has been built since that time has ever been overtaxed, and the sizes are based upon the expectation of carrying off 1 cubic foot of water per second for each acre drained.

The Chairman (R. B. Stanton, M. Am. Soc. C. E.).—I should like to relate to the Society an incident that happened here in Denver in the summer of 1885 of a kind which is not infrequent. Cloud-bursts occur very frequently at the head waters of Cherry Creek. This runs right through the city, but the area drained or any data in regard to it I cannot give. The particular flood occurring in 1885 was photographed during its height by Prof. Richards of the Polytechnic School of Boston, who happened to be here. He took a most excellent photograph of it. The clear water way of Cherry Creek is probably 90 feet in width. The bridges across it are all built 100 feet long, and this water came down in waves of 10 feet in height. Very frequently that creek is filled so that the crest of a wave will be 10 feet from the bed of the creek. Such a flood will come down without a moment's notice, and it would give rather an excessive amount of water to carry through a sewer.

A Member.—Do those cloud-bursts ever occur in the lower part of the State? Has such a thing ever occurred here?

The Chairman.—Yes, sir, to a small extent; not anything like the

cloud-bursts that gather on the divide between the waters of the Platte and Arkansas Rivers.

Mr. GEORGE S. RICE, M. Am. Soc. C. E.—About four years ago I was in the southern part of Arizona, in Cochise County, and I noticed then the effect of one of these cloud-bursts, which are common there. There was a section of country about 2 000 feet long and about as wide. One afternoon a cloud-burst struck this particular spot, on the side of a hill. I was living in the vicinity and visited the spot afterwards. A stream of water from this cloud-burst rushed down in a body about 75 feet wide and about 2 feet high in the deepest place. This temporary stream passed within a short distance of a building made of adobe, and after it left this particular locality, which is about 5 000 feet above the level of the sea, it flowed down into a gulch where there were five or six adobe buildings, and they were swept completely away. That is the way these cloud-bursts sometimes happen, the rain seeming to drop right down from the sky.

Mr. THOMAS C. KEEFER, M. Am. Soc. C. E.—I would like to ask if the irrigation works in this vicinity have suffered from these cloud-bursts, or has special provision been made to meet them? Perhaps some gentleman here could tell us that.

The CHAIRMAN.—Very frequently they are washed away for large distances by cloud-bursts, and it takes a great deal of work to repair them and build them over. The ditch that supplies the City of Denver with water for its trees is washed away in that manner from five to six times in the year. If you will look at the article on the subject of irrigation in the pamphlet prepared for your information you will find some discussion on that very subject.

Mr. RICE.—I would like to relate another incident that I saw. I was located about 10 miles from Tombstone, and in the Tombstone district of Arizona. I attempted to keep a record of the rain-fall and of the temperature in that particular section. We had some alfalfa fields and were hoping that a rain would fall and help us in raising the crop, I noticed that for a week, in a section of country 5 or more miles in diameter, we had no rain, although the country around us had been plentifully supplied. Within this circle the dust was 3 to 4 inches deep on the roads, but go a few miles in any direction and you would find the effect of heavy rains. That is only an instance of how in the same locality you will have a great variation often in the rain that falls. I thought it was rather an interesting point to bring up, although it was not directly connected with this paper.

Mr. JOHN F. BARNARD, M. Am. Soc. C. E.—The trouble is that cloud-bursts take place when the parties are not there and there are no instruments to measure them. That is the trouble all through this country. You have a cloud-burst on the divide between here and the Arkansas, and the next one may be 50 or 100 miles in the other direction.

Mr. Charles Latimer, M. Am. Soc. C. E.—I met one of the citizens of this State the other day, who is a railroad man, and he said that when he came to Salida there was no rain for cultivation, but that now good crops are raised very much farther west than Salida, and that the belt had extended 300 miles westward. Doubtless there must be statistics here which would either prove or disprove that. He was very positive of it.

Mr. Eliot C. Clarke, M. Am. Soc. C. E.—I have lately been a member of a commission appointed to design a conduit of sufficient capacity for carrying off the maximum flood from an area of about 12.7 square miles, in the vicinity of Boston. In considering the subject we examined the various formulas which profess to give maximum rates of flow from given areas. Among these formulas were most of those mentioned in the paper under discussion. The results given by the formulas varied from less than 600 cubic feet a second to nearly 6 000 cubic feet a second. In one case, evidently, no dependence could be placed on the formulas; and yet I suppose that all of them were derived from observation and experience, and, probably, each was applicable to the locality and conditions where it originated. Therefore I think that when such problems are to be solved we must not give too much weight to observations and results obtained elsewhere, but must study with great care the special conditions affecting the locality under consideration.

Mr. F. Collingwood, M. Am. Soc. C. E.—In reference to the question of extension of rain-fall, Dr. Newberry, of Columbia College, who has traveled very extensively over the plains making geological observations, says that that subject must be studied over a series of years or else very erroneous opinions will be formed. He says that you will find the forest area extending gradually for a number of years, trees growing where there were no trees before and evidence of fruitfulness, and yet there will surely come a series of dry seasons when you will find the whole edge of this forest absolutely killed, because of the lack of moisture, and the region will look almost as though burned by fire. Then the cycle will begin again. As apropos to this paper I would like to read the following extract from the Signal Service report for July, 1886:

"The following statement in reference to the exceptionally large rain-fall at Alexandria, Rapides Parish, Louisiana, on June 16th, 1886, is furnished by the observer at that place, in reply to a letter from this office asking the details:

"With reference to the heavy rain fall, it was correctly measured by me, as I watched the gauge closely; the rain seemed to concentrate and center in this particular vicinity, as the wind changed every forty-five or fifty minutes, sometimes more frequently. This town was eight-tenths flooded, in some portions water was 8 feet deep. The rain came down in sheets so heavy that the eye could penetrate but a short distance. The rain-fall from 5 P.M. June 15th, to 6.40 A.M. June 16th, was 11.09 inches; from 6.40 A.M. to 8.45 A.M. 2.16 inches; from 8.45 A.M. to 12.50

P.M., 8.15 inches; total for the nineteen hours and fifty minutes, 21.40 inches."

Mr. JOHN F. BARNARD.—I think that the records which have been kept for a term of years show very conclusively that the rain-fall has been very steadily increasing and advancing westward. Seventeen years ago I came to the Mississippi River, and the country 100 miles west of that was supposed to be comparatively worthless. As settlers appeared and the ground was brought into cultivation, the soil that was loosened absorbed the rain-fall more and more; the subsequent cropping prevented rapid evaporation, and the moisture rose and formed clouds which otherwise would not have been. Showers fell where they had not fallen before, and then after a while these showers went over into the next county, and finally people went into the next county finding that they could raise good crops there; and so it extended. The records show that the rain-fall both in Kansas and Nebraska has very considerably increased, but more important than that, the rain falling and the moisture evaporating, it comes and goes more frequently. It is not as it used to be—a down-pour and then a dry spell. The rain-fall is distributed more evenly over the entire year. There is not a period of heavy rain-fall and then a long drought. That has been about the way it has been advancing as far as I have observed it for seventeen years. I have no doubt that the same influences will be brought to bear upon the eastern counties of Colorado. As Kansas is more cropped and as the soil is more completely stirred up I think the same results will follow. I think that the eastern borders of the State of Colorado will be affected in the same way, judging from the experience in Kansas and Nebraska. I might have prepared myself, if I had thought of it, with statistics from the authorities in the State of Nebraska, showing what the increased rain-fall has been from observations taken for fifteen or eighteen years.

Mr. JACOB BLICKENSDERFER, M. Am. Soc. C. E.—It is stated that the cultivation of the soil and the raising of crops in what was formerly a comparatively arid country, produces an increase of rain-fall. Well, I do not wish to call it in question, at least now; but I would like to ask, is the same observation true in other parts of the country as well as in these western States? Do the same causes produce likewise an increase of rain-fall, and if not, why not?

Mr. LATIMER.—I notice on the line of railroads with which I have had to do, that wherever you cut the trees off, it dries up the springs. In several cases we have had to abandon springs which were formerly very plentiful. The question would be, then, would it not be reasonable to suppose that if we plant trees we will hold the moisture in the ground, and then there would be the evaporation which would bring the water back again? As the former gentleman said, it is a well-known fact that by cultivation you break up the hard covering of the soil, which is like an adobe, and you will have an increased rain-fall all the time.

gentleman, the other day, speaking of the ditches around Denver, said it would have been a very excellent thing if the authorities in granting the right to make these irrigation ditches had required trees to be planted on both sides, as every tree that is planted increases the possibility or probability of rain-fall, or you might say the certainty of it.

The CHAIRMAN.—I wish to make one statement in connection with Mr. Blickensderfer's remarks. There is one fact noted in the cultivation of the plain through which you passed last night—that the cultivation of last year was the first in a large district, and this spring the crops planted on the soil that was broken up last year, prospered very well during the early part of this year, whereas the crops put in this year on new land were very largely killed; also that while there was no rain on that territory this spring, during a certain length of time, yet there was vastly more moisture in the ground broken up last year than in that broken up this year. This bears out the point that Mr. Blickensderfer presented—was the rain-fall increasing or was the moisture retained longer in the ground ?

Mr. BLICKENSDERFER.—I confess to a good deal of skepticism as to the increase of the rain-fall by the cultivation of the soil. Of course, I may be mistaken, and I shall be very happy to be found to be mistaken, because, of course, this country would be benefited largely by an increase of rain-fall ; but it seems to me we scarcely look over the ground with sufficient breadth to cover the whole question. While, of course, I admit it is much easier to ask questions than to answer them, I would like to ask whence comes our rain-fall ? What is the original source of it, and unless that source is increasing, is the rain-fall increasing in the aggregate ? Let us hear something on this subject. As to local observations, of course, we may expect oscillations. There is scarcely anything in nature that does not oscillate more or less. But that does not answer the question. We want to know whether there is a permanent advancement upon the whole or not; not whether there is an oscillatory movement that goes backward and forward and leaves us in the end where we were before. I will just state a matter that came to my knowledge when I was in Salt Lake Valley some years ago. I first made my acquaintance with that valley in 1868. The same question was then up in regard to Salt Lake Valley, and it was urged strongly, especially by those interested in the success of the valley, that the rain-fall was increasing; that it was constantly increasing; that crops could be raised on soil that would not produce a year ago, and as an evidence of increase was adduced, a fact that was undoubted, that the elevation of Salt Lake was constantly increasing. The water was rising. It was spreading over territory which some years ago had been uncovered. You will ask me for the facts. They were pointed out in this wise. "Here," said they, "you see that line of fence." "Yes, sir." "You observe the water is within a foot of the top of that ?" "Yes, sir." "That fence was built

to inclose a meadow which the water did not reach by many hundred yards years ago. Now you see it is submerged, not only for a month, but gradually the water has been rising and coming up all the time since the settlement of the valley up to 1868; and is not that an evidence of an increase of rain-fall—that the rivers flow more water into the valley than they used to ?" Of course we had to admit it as far as it went. Still I own I was skeptical. I left Salt Lake Valley in 1869. I did not get into it again to become much acquainted with it until 1880. What was the condition then ? Salt Lake had gone on to rise for several years after I was there and attained an elevation of nearly 2 feet, or somewhere in that neighborhood higher than when I left, and now it was down almost as low as it was when the Mormons first settled. It has been gradually subsiding for the last few years, I believe, up to the present hour. Now, I do not know what the registers of the Smithsonian Institute give as the rain-fall in this series of years, nor do I know what they give in Nebraska, but while I will very readily admit that if you break up the soil it will preserve more of the rain-fall and convert more of it to useful effect in the production of crops than where there is a hard surface for it to run off, yet I confess still some skepticism as to an actual or permanent change in the record of the district.

One question I would like to have settled in that connection is the one I asked before—whence comes our precipitation, and what are the conditions that govern it now as compared with those that have governed it heretofore, or are likely to govern it hereafter ? As to the effect in the eastern States, it is an admitted fact, patent to every observer, that the denudation of the country of forests reduces the maximum flow of the streams in the summer season, and at the same time increases the maximum flow in times of flood, and it seems to me that it requires very little reflection to see why this is so. It is a fact patent which I can produce reasons for if you please. I know a certain place, for example, where floods in a large valley, one of the main affluents of the Muskingum River, have been registered since 1798; that is the excessive floods, and for twenty years every successive extreme flood has been a little higher than the previous one by the same permanent mark on a certain tree standing on the banks of the stream. Is there a change in the permanent precipitation ? I confess I doubt it.

Mr. Latimer.—As I understand it, there is no difference. There is just as much rain-fall, but it is diffused by the cultivation of the ground, and by holding the moisture in the ground. That is the point. When the ground is packed hard it will run off quickly. In some parts of the country, especially in Arizona, there have been immense territories which have been passed over for many years, people thinking that there was no water at all; and as I suppose the gentleman from Arizona knows very well, they have carried the water in wagons from 50 to 75 miles and sometimes 100 miles, imagining there was no water in

the country. Now, when the Atchison, Topeka and Santa Fé went through they found an abundance of water at 5 feet, but it was below a very hard surface.

A MEMBER.—Are we to understand from this that the rainless belt is not rainless—that water falls on it ?

Mr. RICE.—I would state in answer to the question just asked that it sometimes happens they have six and eight months without any rain. I have known an instance where we would have three weeks without a single cloud in the distance. They have their rain-fall in the summer time, generally in July, August and September. One year I recollect there was no rain from September until the following July. When it did rain it rained heavily, and there were cloud-bursts.

S. WHINERY, M. Am. Soc. C. E.—As a small contribution to the scanty statistics of excessive rain-falls, I submit the following from my own observation. At Somerset, Ky., about noon of August 12th, 1886, a sudden down-pour of rain occurred. The cloud appeared in the northwest and had passed around by the north to the northeast, but suddenly veered and came up from that quarter, accompanied by heavy wind and thunder and lightning. From the beginning to the end of the shower was about thirty minutes. Total rain-fall, 1.57 inches. I estimated at the time that fully four-fifths of this amount fell in twenty minutes, or at the rate of $3\frac{3}{4}$ inches per hour.

At the same place, on August 27th, 1886, about 3 P.M., a small rain-cloud came up from the northeast. It seemed to gather volume rapidly after the rain began falling. There was no breeze at first, but a strong wind soon sprang up from the northeast which veered around by south to the southwest. Total rain-fall, 1.18 inches. Duration of rain-fall, forty-five minutes; but for the first five and the last ten minutes it was a mere drizzle, and I think that fully $\frac{95}{100}$ of the whole amount fell inside of thirty minutes. This would indicate a rate of about $2\frac{1}{4}$ inches per hour.

During the night of May 7th, 1882, a most remarkable rain storm occurred in southeastern Mississippi, on the line of the New Orleans and Northeastern Railroad, then in course of construction. The storm extended from Meridian to an unknown distance southward, but it reached its greatest intensity between the present stations of Tuscanola and Eastabutchie in the Valley of Leaf River. As nearly as could be ascertained the storm continued about six hours, but it was only very violent for about four hours. With the exception of my own rain-gauge at Meridian, no accurate gaugings were obtainable. At Meridian the total rain-fall was $\frac{51}{100}$ inch. At Ellisville, the county seat of Jones County, a cylindrical tin can 8 inches deep, used by an assistant as an improvised rain-gauge, was filled, and overflowed an unknown amount. Rocky Creek, 2 miles south of Ellisville, with a drainage area of about 15 square miles, rose to a flood cross-section of about 4 700 square feet. Its velocity could not have been less than $3\frac{1}{2}$ feet per second, giving a dis-

charge of about 16 500 cubic feet per second. If we estimate that two-thirds of the water falling flowed off the ground into the creek (the whole valley is open pine woods; soil a very sandy loam, covered at the time with thick grass about a foot high), we will find that a rain-fall of 2½ inches per hour was required to produce such a flood. In Leaf River Valley, between the stations named above, the country is sparsely settled by a class of people not accustomed to making accurate observations, and it was difficult to obtain full and reliable information. Pains were taken to get all the facts possible, and after making abundant allowance for exaggeration, I was forced to believe and report that not less than 16 inches of rain fell during the storm. Without detailing the evidence that led me to this conclusion, I may mention a few facts obtained. One of the most intelligent and reliable residents of the neighborhood stated that a coal oil barrel with the head out, standing in an open lot, which barrel he was certain was empty when the rain began, was fully half filled in the morning. The small streams in the vicinity reached a flood volume which could not be accounted for with a rain-fall short of 4 inches per hour. Near the edge of a gently sloping table-land, between two small streams, but above their flood level, green pine logs 30 inches in diameter were floated and moved from their beds. Considering all the circumstances, I do not think it improbable that the rain-fall during the most violent part of the storm reached a rate of 5 inches per hour.

The paucity of records of excessive rain-falls is greatly to be deplored, and equally lamentable is the fact that no adequate effort is being made to supply the deficiency. It will require, at best, almost a generation to collect the necessary mass of statistics from which the general laws and limitations of excessive rain-falls can be deduced. Aside from a few chance and isolated observations, we are practically doing nothing in this field now. The Signal Service Department of the Army is undoubtedly the proper agent for observing and collecting such data, but there seems to be no systematic effort in this direction, and out of all the Signal Service stations in the country, only one is equipped with a self-registering rain-gauge. There are, I believe, a very small number of such instruments in the hands of amateur observers and institutions of learning throughout the United States, but if the records of these are published at all, they are not readily accessible to engineers. Without such automatically recording instruments satisfactory observations are not possible. In using the ordinary rain-gauge for this purpose, the observer must always trust his judgment, more or less, as to the amount and duration of down-pours that may occur during a continued rain storm. The Government should be urged to substitute such self-recording instruments for the ordinary rain-gauges at all its signal stations. The instrument-maker who will bring out a reliable self-registering rain-gauge, the cost of which shall be within the reach of ordinary

amateur observers and engineers, will contribute largely toward securing the co-operation of private observers in this field. Such an instrument need not be perfectly exact as to time and quantity—if it gives results correct to the nearest $\frac{1}{10}$-inch of rain-fall and five minutes of time, it will serve all practical purposes.

Of almost equal importance to a knowledge of the maximum rate of precipitation in any locality is that of the frequency of recurrence of rain-falls of a given rate and amount; for even those engineers who think it necessary and wise economy to provide, in city sewerage, for the heaviest possible rain-falls, will admit that in railroad and highway practice it may be bad economy to design culverts to carry floods that only recur once in thirty or forty years. The fact that Captain Hoxie has been only able, in making a special study of the subject, to collect the meager data given in the table of excessive rain-falls in this paper, shows how very small is the available stock of information on the subject.

There are, doubtless, in existence, scattered about in the note-books of engineers and amateur observers, a great many other records of value, but they have never been collected together, and are not therefore available. The importance of the subject is sufficient to warrant the Society in taking some official action, not only in the matter of collecting such scattered records, but to urge on the Chief Signal Officer the importance of systematic attention to the collection of such data, and of using self-registering rain-gauges at every station.

While there is evidently no direct relation between the maximum rate of precipitation and the mean annual rain-fall of any place, it is probable that the former bears some rude ratio to the latter. In the absence of any facts to the contrary, it would not be reasonable to expect and provide for the same rate of precipitation in a region where the annual rain-fall is 30 inches as where it is 40 inches. And inasmuch as the engineer is often called on to design sewers and culverts in localities where the mean annual rain-fall is known, but where no records whatever of maximum rain-fall can be obtained, a rude approximation of that ratio may sometimes be useful. An inspection of the few records we have seems to indicate that the following empirical formula may be safely used:

$$r = \frac{R}{12}$$

where R = mean annual rain-fall in inches.

r = probable maximum rain-fall, in inches, for one hour.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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490.

(Vol. XXV.—July, 1891.)

MOUNTAIN RAILROAD CONSTRUCTION.

By WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E.

Although the fallacy of the narrow gauge arguments advanced so vehemently a few years ago is now generally understood, still railroad men generally do not appreciate the fact that the same results in the way of light construction, sharp curves, and, of course, heavy gradients, can be obtained with a standard gauge railway almost as well as with a narrow gauge, and without the attending disadvantages.

To illustrate this the author presents a description of some extensions that were added to a system of lumber railways in Northwestern Pennsylvania, under his direction, during the past year. This system, which will aggregate about 70 miles, is intended to develop the fine hemlock forests of that region. Topographically the country is much broken, being intersected by a number of small streams, from which the hills rise abruptly. It became necessary, therefore, that each principal valley should have a line of railway with tram-road feeders from the several mills located on the branches. A rather large mileage for the territory to be served was therefore required, on which account it was necessary to keep the expense of construction as low as possible. The heavy timbering and hardness of the material to be moved were constant obstacles to cheap work, while the ruling gradient in each case was rendered so very steep by the elevations to be overcome in

short distances that no breaking of gradient line was permissible, and therefore heavy work in rough places could be avoided or lightened only by the introduction of sharp reversed curves.

To illustrate the alignment and work, a small portion of the profile and map of one of the main stems is shown. (See Plate XV.) This particular line was 7 miles long, running up one valley, crossing the summit, and then down a stream on the other side a total distance of 7 miles, although 2½ miles are to be added as a future extension.

From the bottom, where connection was made with an existing track, to the summit was 3.64 miles, with difference in elevation of 606 feet. A compensation of 0.03 was decided on, so that the gradients on tangents worked out to be 3.316 per cent. or 175 feet per mile. The gradient as shown here was in favor of the traffic, but on the other side of the divide where the country was quite similar, a gradient of 3.182, compensated, had to be used against the loads.

The limit of curvature was put at 18 degrees, although once in heavy work on the other side of the hill, one curve of 19 degrees was laid out. As speed would under all circumstances be low, super-elevation could be kept down, and therefore a minimum length of tangent of 100 feet between curves was permitted, although in a few instances shorter tangents had to be used, the shortest being 77.1 feet between the 18-degree curves at station 135 as shown on the map.

Surveying of these lines was slow and expensive owing to the heavy timber. The difference of levels between summit and starting points was determined as closely as possible in the usual manner by aneroids, and then a trial preliminary run. It was afterward found much more expeditious to equip the chief of party with a light tripod-mounted compass provided with a telescope carrying a long bubble tube and vertical circle. By setting the circle at the gradient angle he laid out the preliminary in advance of the party, so that the axmen could clear the line of obstructing trees. Moreover, the line as thus staked closely approximated the actual grade line. Topography was accurately taken, plotted on a scale of 200 feet per inch, with 5-foot contours (or 100 feet per inch in rough places), the line of grade projected, and the railroad line fitted as closely to this as 18-degree curves would allow. On running this line in the field the changes required were but slight. In a heavily timbered country and where cost of work must be closely watched, the method of "paper" location is incomparably superior to field location,

which, under such conditions, at least, is little better than a good guess. The cost of engineering for location and construction, including running complete preliminaries on two different routes, was about $400 per mile. A large portion of this expense, however, was due to the number of axmen rendered necessary.

Cuts were made 16 feet wide, and fills 12 feet. The whole of the work being on side hill, which in many places was very steep, and in a wet country, much water had to be dealt with. Cross drain-boxes were used freely. Culverts under 4-foot banks were single or double wooden boxes 12 x 24 inches; under 10-foot banks, log culverts were adopted; while under banks over 10 feet, dry rubble culverts 2 x 3 feet, single or double, were built. There were on the 7 miles of the line, part of whose profile is shown, 75 271 cubic yards of material moved, of which 5 633 cubic yards, or 7.6 per cent. were solid rock; 8 298 cubic yards, or 11 per cent. loose rock; 26 468 cubic yards, or 35.2 per cent. tough clay where six picks were required for five shovels; and 34 872 cubic yards or 46.2 per cent. earth, most of which was heavy soil. The actual cost of labor, explosives and tools, including also feed and care of horses and mules for graduation, was $35 938.22, from which there is to be deducted a credit for value of tools left on hand of $750, so that net actual cost was $35 188.22. This covered: grubbing and clearing 51 acres; doing all the earthwork, including slope ditches; 467 cubic yards dry culvert masonry; sixty-eight wooden box and log culverts; and 2 424 feet of hemlock lumber for drain-boxes; in fact everything complete, ready for track, except trestles. This is an average of $5 026.89 per mile, or $0.465 per cubic yard of earthwork, without regard for classification, including, however, the cost of the sundry extras mentioned above; while the great percentage of rock and hard pan is to be borne in mind. The right of way was 50 feet wide, and as the fills and cuts were light, the proportion to be grubbed was great. This work, under such conditions, cost between $50 and $60 per acre.

In light work of this character, especially where the country is timbered so that roots interfere with continuous burrowing and where soil is heavy, there is great danger of underestimating cost of graduation. A generous allowance should be made to ordinarily prevailing contract figures.

The only available wood for trestles was hemlock, which cost from $7.50 to $8 per thousand feet. Large sticks were easily procured, and

on account of this and the low price it was possible to have the dimensions of the stringers greater than is usual with pine, in order to allow for the unreliability of hemlock. Bents were set 12 feet between centers. Posts and sills were 12 x 12 inch timbers, caps were 12 x 12 and 12 x 14, while stringers were single sticks, 12 x 18 inches. Bents were stayed laterally and longitudinally by 12 x 2 inch plank bracing. All members were secured by drift bolts. The bents stood on mud sills. 102 258 feet B. M. of hemlock were used, of which 13 704 feet were hewn from trees immediately adjacent to track. The sills and mudsills were generally made from this latter. About $1\frac{1}{3}$ feet B. M. timber were used for each cubic yard of earthwork. Cost of labor per thousand feet for framing and erecting was $9.53, which included, however, expense of felling and hewing 1 142 lineal feet of timber, as mentioned above. Cost of bolts and spikes was twelve cents per lineal foot of trestle.

Average cost per mile of trestles, varying in height up to 28 feet:

12 650 feet B. M., at $7.75	$98 04
163 lineal feet hewn timber. No charge.	
Bolts, spikes, etc	13 20
Labor, hewing, framing, erecting	130 21
Total per mile	$250 45

As to superstructure. The gauge was 4 feet $8\frac{1}{2}$ inches. Rails weighed 40 pounds per yard, with angle bars. Joints were supported and broken. Ties were of hemlock, usual size, and spaced seventeen to a rail length. Split switches were used, with automatic switch stands, using the low target pattern, with weighted ground lever as the automatic device. The cost of 1 mile of such track was on last year's prices:

62.86 tons rails, at $33	$2 074 38
352 joints complete, at $0.55	193 60
6 200 pounds spikes, at $0.225	139 50
3 000 cross-ties, at $0.15	450 00
Freight on materials	159 00
Track-laying	400 00
Total	$3 416 48

The average cost per mile of the most expensive section which is represented by profile was:

Graduation	$5 026 89
Trestles	250 45
Track	3 416 48
Engineering	400 00
Total	$9 093 82

This represented the heaviest work and consequently most expensive line; on other lines where the topography was lighter the cost of graduation was lightened, so that the total cost per mile was reduced to about $6 500. But these limits of figures furnished a railway, except as to unusual alignment and lightness of superstructure, standard in all respects. With a limit of curvature of 10 or even 12 degrees, the building of these lines would not have been feasible.

The switch-back method of line development was used. One switch-back was constructed on a gradient of 3.746 per cent. against traffic, and two more are laid out on projected lines. Switch-backs for lumber roads have one great advantage, that they can generally be laid out near the head of the valley, with the effect of throwing the line down into the valley bottom, so that the timber can come down to the road. They also serve as safety switches on steep gradients.

The motive power consisted of two forms of locomotives. One a patented machine called a "Shay Gear" locomotive (see Plate XVI.), and the other an ordinary Mogul. These Shay gear engines have three cylinders set vertically and together on one side of the boiler, just ahead of the cab. They connect with a shaft which runs the whole length of the engine and tender, and is geared by bevel-gearing to every wheel. Between each wheel or pair of wheels is an articulated joint in the shaft, so that the length of rigid wheel base is that of an ordinary bogie truck. The great advantage obtained is that the whole weight of engine and tender is utilized for traction, and distributed over a much greater length of track than is the tractive weight of an ordinary locomotive. The incidental advantages are short wheel base and high piston speed. These engines are therefore adapted to slow speeds, steep gradients and sharp curves. The disadvantages are the enforced low speed and those attending such mechanical devices.

These engines are manufactured to weigh from 20 000 to 180 000 pounds, all of which is utilized for traction, and with two cylinders 7 x 7 inches for the smallest size, to three 16 x 15 inches for the largest. The maximum rigid wheel base is 56 inches. The gearing ratio is slightly more than two to one.

The dimensions of the engines on the road in question were:

Eng. No.	Weight.	Cylinders.	Size Drivers.
2	120 000 pounds.	3–14 x 12 inches.	12–32 inches.
4	56 000 "	3–10 x 10 "	8–28 "
5	70 000 "	3–11 x 10 "	8–29½ "

A view of one of the Shay locomotives, built for another line, is shown.

The fourth engine in service is an ordinary Mogul of the following dimensions:

Weight of engine, 70 300 pounds; weight on drivers, 61 200 pounds; three 4-foot drivers on each side. Length of rigid wheel base, 10 feet; cylinders, 15 x 24 inches. This engine is in constant service and finds no difficulty in working around the sharp curves, showing that a special form of engine is not necessary for such alignment as described.

As a proof of this, and to show what strain a light track if built with care can resist, the following incident attests. The Mogul engine with seven loaded cars of lumber started down the 3.316 per cent. gradient illustrated. But the brakes were defective and the train immediately passed beyond the trainmen's control. The engine and cars ran down the hill 3½ miles, and through the reverse curves without damage to themselves or track. On passing the junction of the new track with the old line the combination of a flat curve out of line and a sag in the grade threw one car off the rails, when all the cars capsized, the engine, however, remaining on the track.

The author does not advocate the use of sharp curves or light rails, but cites the construction and operation of this road to show that under proper restrictions sharper curves and lighter rails can be used than is generally supposed, and so render possible the construction of railways that would not be feasible if the ordinary limits of curvature and superstructure were enforced. Speed is much more destructive to track than weight, and the constantly increasing weight of rails is rendered necessary more by the increase in speed of trains than by that in the weight of the locomotives. If speeds are kept low a light weight of rail will be found serviceable.

In the construction of such railways as much (or even more) engineering skill will be required as in the building of more pretentious lines. Every foot of the alignment must be carefully studied, and every item of expenditure rigidly considered, so that the construction becomes a connected series of small detail problems.

PLATE XV
TRANS. AM. SOC. CIV. ENG'RS.
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PARSONS ON
MOUNTAIN RAILWAY.

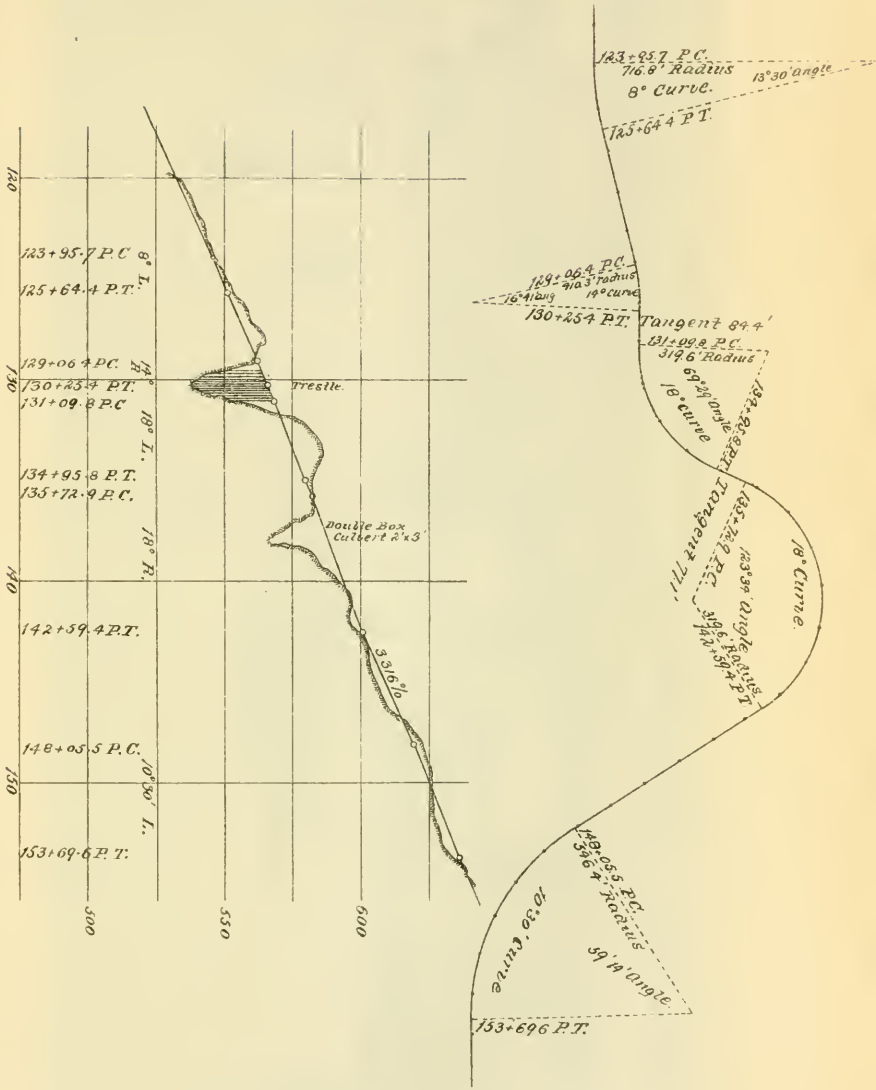

PLATE XVI.
TRANS. AM. SOC. C. E.
VOL. XXV, NO 490.
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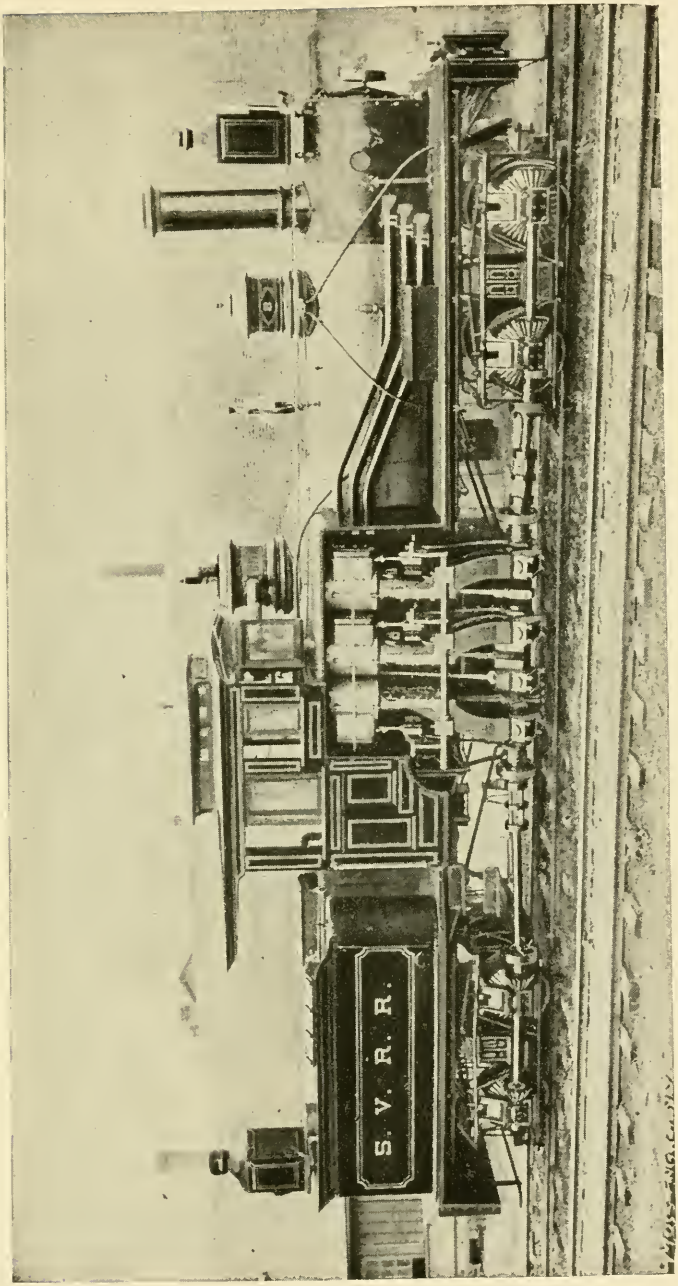

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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491.

(Vol. XXV.—August, 1891.)

INLAND SEWAGE DISPOSAL, WITH SPECIAL REFERENCE TO THE EAST ORANGE, N. J., WORKS.

By CARROL PH. BASSETT, M. Am. Soc. C. E.

WITH DISCUSSION.

When the first official census of this country was taken one hundred years ago (1790) the thirteen States and unorganized territory then embraced a population of 3 929 214. Less than 5 per cent. of these people were found west of the Appalachian Mountains, the densest populations being found along the Atlantic seaboard, in Massachusetts, Rhode Island, Connecticut, and about New York and Philadelphia. The average depth of settlement at right angles to the coast was then 255 miles. The settled area comprised approximately 239 935 square miles, with an average density of 16.4 per square mile. In 1850 the total area had increased to 979 249 square miles, with a population of 23 191 876, and average density of 23.7 per mile. In 1880 the area of settlement was 1 569 570 square miles, containing a population of 50 155 783, or an average density of 32 per square mile. The center of population in 1790 was near the present site of Washington, D. C. From thence it has moved westward, about a degree to a decade, closely following the

39th parallel, until in 1850 it was a few miles south of Parkersburg, West Va., and in 1890, 8 miles south of Cincinnati.

Among the older States of the East the density of population is several times the average density throughout the country. In 1880 the average in Massachusetts was 228.6 per square mile; in Rhode Island, 211.7; in New Jersey, 135.6; in Connecticut, 131.1; in New York, 108.1; in Pennsylvania, 93.1; in Maryland, 84; and in Ohio, 80. As we examine in still greater detail the development of the country, we find in 1790 the frontier line at 255 miles from the Atlantic coast, gradually pushing inland and avoiding the arid and rugged districts. The process of natural selection makes itself felt, and the populations, having to rely almost entirely on water transportation, conform themselves to the river valleys. In our early growth, all large centers of populations were necessarily located on the larger water courses, and although the development of means for overland transportation has largely modified the conditions originally existing, experience shows that large masses of population are still most liable to appear near water-courses. A recent estimate (W. J. McGee, in the *Forum*) states that fully 25 or 30 per cent. of the population of the eastern United States is crowded upon the 14 per cent. of alluvial lowland.

If the three hundred and fifty-five cities recorded by the eleventh census in the States between New England and the one hundredth meridian are grouped according to their situation with respect to water ways, two hundred and four, with an aggregate population of 5 593 340, are found to be riparian. There are fifty-nine seaboard and lakeside cities, with an aggregate population of 6 880 043, forty-four inland cities with a population of 840 466, and twenty-eight unclassified cities aggregating 675 676 in population. Excluding the seaboard and unclassified cities, there remain two hundred and forty-eight centers, with an aggregate population of 6 033 806, of which 89 per cent. is riparian; or, including the various classes, there is a total urban population of 13 989 529, of which 40 per cent. is riparian.

Ordinarily, water for domestic and manufacturing purposes is supplied to these populations from the rivers above cities, or from a lake on the borders of which they are situated. But, from the nature of our civilization, it transpires that enormous waste products are constantly thrown off from the body corporate. Considerations of convenience have prompted the use of water-courses for the final removal of these

wastes. In the development of these tendencies we have come to see along our prominent river valleys populations pouring large quantities of filth into rivers which a few miles lower in their course furnish supplies of potable water to other peoples.

The magnitude of the rivers and the comparative sparseness of the population in past years made these conditions possible, and the evils have developed so gradually that they have escaped the attention they merit. Naturally, the rivers along the middle Atlantic seaboard furnish the best developed illustrations of pollution.

The area of watershed, and the average density of population in some river valleys in the Eastern States, is here given:

Drainage Basins.	Area in Sq. Miles.	Population in 1880.	Per Sq. Mile.
Hudson	13 248	2 280 359	172.1
Delaware	11 362	1 999 921	176.1
Susquehanna	27 655	1 715 009	62 0
Schuylkill	1 870	362 000	200.0
Passaic	949	320 762	338.0

It should be remembered that 3 miles above Philadelphia is Conshohocken, 7 miles further up the valley is Norristown, 17 miles further is Phœnixville, and 29 miles further is Pottstown; all these cities use the Schuylkill water for a public supply and also as a sewer, into which enter whatever foul liquids are voided by these cities. A few miles further up the river, Reading adds its filth, but is prevented from using the river as a water supply, owing to the presence in it of large amounts of mine water. The increase in pollution is evident from the table given on page 128.

The conditions which obtain in these instances are no exception to the general experience of the country. Chicago is wrestling with the gigantic problem. The cities along the Ohio, the Hudson and the Passaic; and in Massachusetts, along the Blackstone, Merrimac, Charles, Sudbury, Chicopee and Concord have felt the evil. Every populated river valley furnishes similar testimony. Streams acting as natural drainage channels are popularly looked upon as the outlets for crude sewage. There appears to be no adequate energy at work to change these conditions. Practically, no effort has been made in this country to

A SUMMARY OF POLLUTION OF THE RIVER SCHUYLKILL BY DOMESTIC SEWAGE, published in the Pennsylvania State Board of Health Report for 1885, from Investigation made in that year, is here inserted for illustration. (Population estimated for January 1st, 1885.)

ITEMS.	DISTRICTS.						
	FIRST.	SECOND.	THIRD.	FOURTH.	FIFTH.	SIXTH.	SEVENTH.
	Whole valley above Reading.	FROM *Upper Boundary of Reading to Mouth of Manatawny Creek.*	FROM *Above Manatawny Creek to Intake Phœnixville Water Works.*	FROM *Phœnixville Water Works to Norristown Water Works.*	FROM *Norristown Water Works to Conshohocken Water Works.*	FROM *Conshohocken Water Works to Roxboro' Pumping Station.*	FROM *Roxboro' Pumping Station to Fairmount Pumping Station.*
Drainage area	656.9 sq. m.	3980 sq. m. 1054.9 "	149.4 sq. m. 1204.3 "	517.6 sq. m. 1721.9 "	29.5 sq. m. 1851.4 "	38.5 sq. m. 1789.9 "	74.0 sq. m. 1863.9 "
Population	91 000	95 000 186 000	28 000 214 000	66 000 280 000	22 000 302 000	18 000 320 000	52 000 372 000
DOMESTIC SEWAGE—				†			
Daily water supply* representing domestic waste water.	2 600 000 gals.	4 500 000 gals. 7 100 000 "	200 000 gals 7 300 000 "	500 000 gals. 8 800 000 "	1 000 000 gals. 7 800 000 "	80 000 gals. 8 880 000 "	
Population having water-closet drainage into river	5 000	12 000 17 000	750 17 750	1 100 18 850	2 800 21 650	1 100 22 750	4 150 26 900
Population having wash water drainage into river	22 000	40 000 62 000	5 000 67 000	3 000 70 000	4 000 74 500	1 500 76 000	9 000 85 000
Free ammonia	0.0035	0.0040	0.0025	0.0036	0.0010	0.0015	
Alb. ammonia	0.0140	0.0085	0.0100	0.0075	0.0140	0.0100	
Oxygen reg'd	0.058	0.14	0.19	0.13	0.23	0.15	
Nitrites	.....	0.00005	0.00005	0.00008	0.0005		
Nitrates	0.96	0.92	0.92	0.86	0.86	0.80	
Chlorine	0.20	0.25	0.25	0.25	0.06	0.20	
Hardness	2.5	3.5	3.70	.....	.....	3.0	
Total Solids	9.0	11.5	10.5	10.5	10.0	11.5	

Figures underlined indicate totals down to the lower end of the district represented by the column in which they occur.

* From public supply only.

† Perkiomen watershed above Schwenksville not included in the remainder of this column.

purify sewage where a stream of large volume offers available outlet. Present tendencies may be roughly outlined in a few words.

Many cities in populated river valleys are abandoning their river supplies, as admittedly unsafe, or are actively debating the problem of procuring water, at great expense, from distant and undeveloped highland (*e. g.*, Albany, Syracuse, Newark, Philadelphia). Inland towns, to which no neighboring stream offers possible outlet for sewage, are combining under State supervision (notably in Massachusetts), or by mutual agreement, to secure through union of interest an outlet for crude sewage into some distant water-course. It is not contended that such action is unjustifiable, or even that it is unwise under some conditions; but the practice diverts attention from different methods, in other places essential. Towns, unable to bear these expenses, are continuing on the one hand to use polluted water for potable purposes; or on the other, are storing filth in their soil and neglecting to provide sewers; because collected sewage requires some purification, which they are unwilling to attempt. Public knowledge of the true economy of stream conservation and the methods available for sewage purification, is insufficient to correct these tendencies.

With few exceptions, the laws governing stream pollution in the several States are inadequate either in their intention or methods of enforcement. A high court of New Jersey has recently endorsed the opinion that the sewage from a population of 15 000 can enter a stream having a minimum daily flow of 125 000 000 gallons, already largely polluted, and flow four miles on a level reach before entering the public water supply of 400 000 people, without demonstrated danger. The development of the germ theory of disease and the proofs on which it rests, seem likely to overturn such opinions, and it appears highly probable that demonstrated scientific knowledge will support the natural aversion of the senses, to such an assumption. Recent comparative investigations of zymotic diseases in towns using sewage-contaminated river water and those using water free from sewage inflow, furnish strong presumptive evidence that sewage-contaminated water supplies and disease. are related as cause and effect. With the germ theory for a premise, no other conclusion is reasonable. It is hoped that prophylactic scientists will accumulate these evidences and press them to their legitimate conclusions.

The conclusions reached by the Water Supply Committee of the

Massachusetts State Board of Health during the past year is here relevant. "The two cities—Lowell and Lawrence—both well situated, well regulated, and comparing favorably for general healthfulness with other cities in the State, have 50 per cent. more deaths by typhoid fever, for the same population, than any other cities in the State. These are the only two cities in the State which draw their water for drinking from a river into which, within 20 miles above, sewage is publicly discharged."

The natural purifying power of streams is realized, but the safe limits of pollution are by no means understood. This limit, beyond doubt, has been reached in several streams of the country now serving as potable supplies. It is not proposed here to discuss the permissible limits of pollution in streams, or the natural processes of purification, which are actively combatting the evil. Reference is made to the following papers:*

Aside from the large number of cases where crude sewage is discharged into streams with detriment, the need of sewage purification is being felt in a growing class of inland cities isolated from any considerable bodies of water. Residence towns are likely to develop under these conditions with increasing rapidity in the future. Sewerage, if accomplished at all in these communities, must be accompanied with some purification of the sewage.

Few papers having been presented before the Society dealing with the subject in hand, the author may be pardoned for here inserting, at the expense of repeating much that is not new, some general statements regarding present practice in sewage purification, which bear indirectly on the special works here to be described at length.

Sewage purification processes, as successfully operated at present, may be grouped under the heads of—(*a*) Irrigation; (*b*) Filtration; (*c*) Chemical Precipitation, or a combination of these processes. Electrical processes are as yet barely fortified by practical experience.

(*a.*) For irrigation, the sewage is conducted in carriers or open channels and distributed, preferably intermittently, over land. Not only is

*Charles G. Currier on "Self Purification of Flowing Water and the Influence of Polluted Water in the Causation of Disease," with its discussion, in February, 1891. Transactions Am. Soc. C. E.

Charles C. Brown on "River Pollution in the United States," before the Engineers Club of St. Louis, 1890.

Rudolph Hering, "Notes on the Pollution of Streams," read at Am. P. H. Assoc., November 10th, 1887.

purification secured, but successful utilization of the manurial components of the sewage may, in many cases, be obtained from crops raised upon the surface. When large tracts of suitable soil are available, the sale of the crops may materially reduce the cost of manipulation.

Under-drainage of the land is necessary where a compact soil is encountered. As the temperature of sewage at the outfall seldom falls below 40 degrees Fahr., little difficulty need be feared in this latitude from the freezing of the ground if it is porous and properly manipulated. The character of the sewage and the nature of the soil are important elements in determining the amount of ground necessary for irrigation. Approximately, one acre of ground for every one hundred inhabitants contributing to the sewage must be allowed, where utilization is proposed. A city of 1 000 000 inhabitants would thus require 10 000 acres. Near large centers of population such immense tracts are seldom available. At Berlin, Germany, however, which is surrounded by sandy plains, the sewage is pumped to extensive farms from 6 to 12 miles away, at Osdorf, Frederikenhof, and Falkenburg, and successfully purified. The sandy character of the ground is there well adapted to the disposal of sewage, its area is hardly limited, and the financial returns fair. Other good examples of irrigation are found; in Germany, at Dantzig and Breslau; in France, near Paris, at Gennevilliers; in England, at Croyden, Warwick, Bedford, Doncaster, Oxford, Leamington and Wrexham; in this country, at Pullman, Illinois.

Following the processes of purification in sewage subjected to land irrigation, we find the soil first acting as a mechanical filter, the coarser matters in suspension being retained on the surface. The liquid containing the dissolved matters and the smaller particles in suspension descends through the ground, and is divided into myriads of particles with an enormous aggregate surface, favorable opportunity for oxidation occurring when the soil is kept aerated. The scavenger bacteria (*bacterium termo*, most frequently), there inaugurate nitrification; oxidation or combustion of the organic impurities being coincident. Professor Warrington, in a paper read before the British Medical Association, in Montreal, in 1884, was among the first to record the result of experiments with these bacteria and note their methods of work. The function of the bacteria is to pull apart the more complex tissue of higher organisms, and resolve it into its original elements. They thrive ordinarily only to a depth of from 2 to 3 feet from the surface. It is unfortunate that

our information regarding these allies is so meagre; but careful work of investigation among able biologists in this country and in Europe, may be relied on to materially increase our acquaintance in the near future.

(*b.*) The scarcity of ground near large centers of population has suggested a modification of ordinary irrigation processes, known as "Intermittent Downward Filtration." The land to be irrigated is thoroughly under-drained at depths from 4 to 6 feet. The sewage should be applied to the land intermittently. The surface is flat or laid out on the ridge and furrow system and as even as possible, to secure uniform distribution and filtration. The sewage of about 500 people can be purified on one acre of highly porous ground properly prepared. Purification rather than utilization is the aim of this process. The process is in use in many English towns. Mr. Bailey Denton first carried the process to great perfection on a large scale at Merthyr Tydvil. In this country, the process is in operation at Lenox, Medford, South Framingham and Sherborne Reformatory in Massachusetts; Cranston, R. I.; Norristown, Pennsylvania; Morris Plains and East Orange, New Jersey.

The application of crude sewage to a small area of ground soon coats the surface with a film of putrescible matter liable to decomposition, which closes the pores of the ground. By the interception of the matters in suspension this difficulty can be avoided. For this purpose, subsidence tanks are sometimes introduced, but their effect is slow and superficial. Mechanical filters of sand, gravel, coke, charcoal, burnt earth, iron, slag, and other substances, produce results better than simple subsidence, and may be made of service for partial clarification. It is difficult to keep the media clean for efficient filtration.

Upward straining through coarse media has merits where the larger matters in suspension, only, are to be removed.

An additional modification of land irrigation has been extensively introduced in this country, from England, for the disposal of the sewage of private houses or public institutions on land immediately adjoining. The process was originally suggested by Rev. Henry Moule, in 1868; was further developed by Mr. Roger Field, in England, and was first prominently advocated in this country by Colonel G. E. Waring, Jr. Open-jointed tile are laid from 8 to 12 inches below the surface, and into these the sewage is flushed (best by an automatic syphon) and allowed to soak into the ground, oxidation occurring as in other irrigation pro-

cesses. The process is in use at many large public institutions, residences, hotels, and is advocated for the disposal of small volumes of sewage, where cesspool, storage, or direct stream pollution is otherwise liable to occur. Excellent illustrations of this system may be seen at Morris Plains and Lawrenceville, New Jersey, and Bryn Mawr, Pennsylvania.

(c.) It remains to consider processes of chemical precipitation. Many of the faults and objections raised against this process are as weak as were the extravagant advantages claimed for it a few years ago in England. The author's belief is that in some form of chemical precipitation lies the great hope for sewage purification, in compactly populated districts. The precipitation of matters from solutions has long been recognized as a possibility by chemists, and profitably applied in manufactures. An infinite variety of chemical precipitants for sewage treatment has been suggested, but the essential details of all are identical.

The selected chemicals are thoroughly mixed with the sewage, after which the liquid is brought to practical rest in large tanks, where precipitation occurs. The process is first mechanical, the heavier matters settling by their own gravity to the bottom of the tanks; but a second action should be secured by the chemicals, viz., the creation of a coagulating film which entangles and carries down the finer particles in suspension and removes some of the matters in solution. The impurities deposited in the tanks, together with the water retained by them, form a dark, viscous mass, containing from 90 to 95 per cent. of water, technically called "sludge." The greatest difficulty experienced in chemical precipitation is in disposing of this sludge. Absorbents are sometimes added to render it portable. The method of manipulation by filter-presses has given probably the best results, and will be described more fully herein.

Cheapness and availability of the chemicals used in precipitation are matters of prime practical importance; to these considerations is largely due the general use of lime, salts of iron, and alumina. Blood, charcoal, clay, phosphates, manganates, and chlorine are among the other chemicals that have been used. It has been found possible in England to secure an effluent from sewage purification works, where a chemical process only is used, which conforms to the standard of purity erected by the River Pollution Commissioners.

A process known in England as the "International," and approved

by Professor Frankland, uses for a precipitant a patented compound called Ferozone (above 3 grains per gallon), and the effluent filtered through "polarite" is clear and odorless; the sludge is compact, about 40 per cent. of the bulk of the precipitate being from lime. Recent experiments at Salford, England, with this process, showed results about as good as those from an electrical test, and better than those from the albumino-ferric process.

The electrical process with a 50-volt, 50-ampère current, consuming iron at the rate of .46 pounds per 1 000 gallons, produces an excellent effluent and compact sludge. For full description of the electrolysis of sewage, see *Engineering News*, October 26th, 1889, an abstract of an article read before the British Association by William Webster.

Chemical precipitation processes are in operation at Frankfort, Germany; Coventry, Leeds, Hertford, Aylesbury, Tottenham, Layton, Burnley, Bradford, Leicester, Southampton, and other places in England. In this country the first sewage purification works involving a systematic chemical treatment on a large scale, were erected at East Orange, N. J. Worcester, Mass., has since placed in operation, works purifying part of the city's sewage. At Round Lake and Coney Island, N. Y., and at Long Branch, N. J., works for partial precipitation of sewage have also been erected.

It is now proposed to describe the East Orange Works and the steps which led up to their construction. As early as 1881, the citizens of East Orange, who are largely active, intelligent business men of New York City and Newark, began to look about for some method for removing domestic wastes. The town is situated along Newark's western boundary and is surrounded by South Orange, Orange City, and Bloomfield on the south, west and north, respectively. The small streams near the town furnish no available outlet for crude sewage, and land in the midst of an exceptionally well-to-do community was difficult to procure even at high figures, particularly for a matter so æsthetically unpleasant as sewage purification. Outlet through the sewers of Newark on the east was suggested, but without approval by that city, and a pumping-main over the ridge to the south, requiring the elevation of the entire sewage, to extend eight miles across adjoining townships to Newark Bay, was designed. Reference to the accompanying plan (Plate XVII) of the thickly-settled parts of Essex County, N. J., will show the relation of

PLATE XVII
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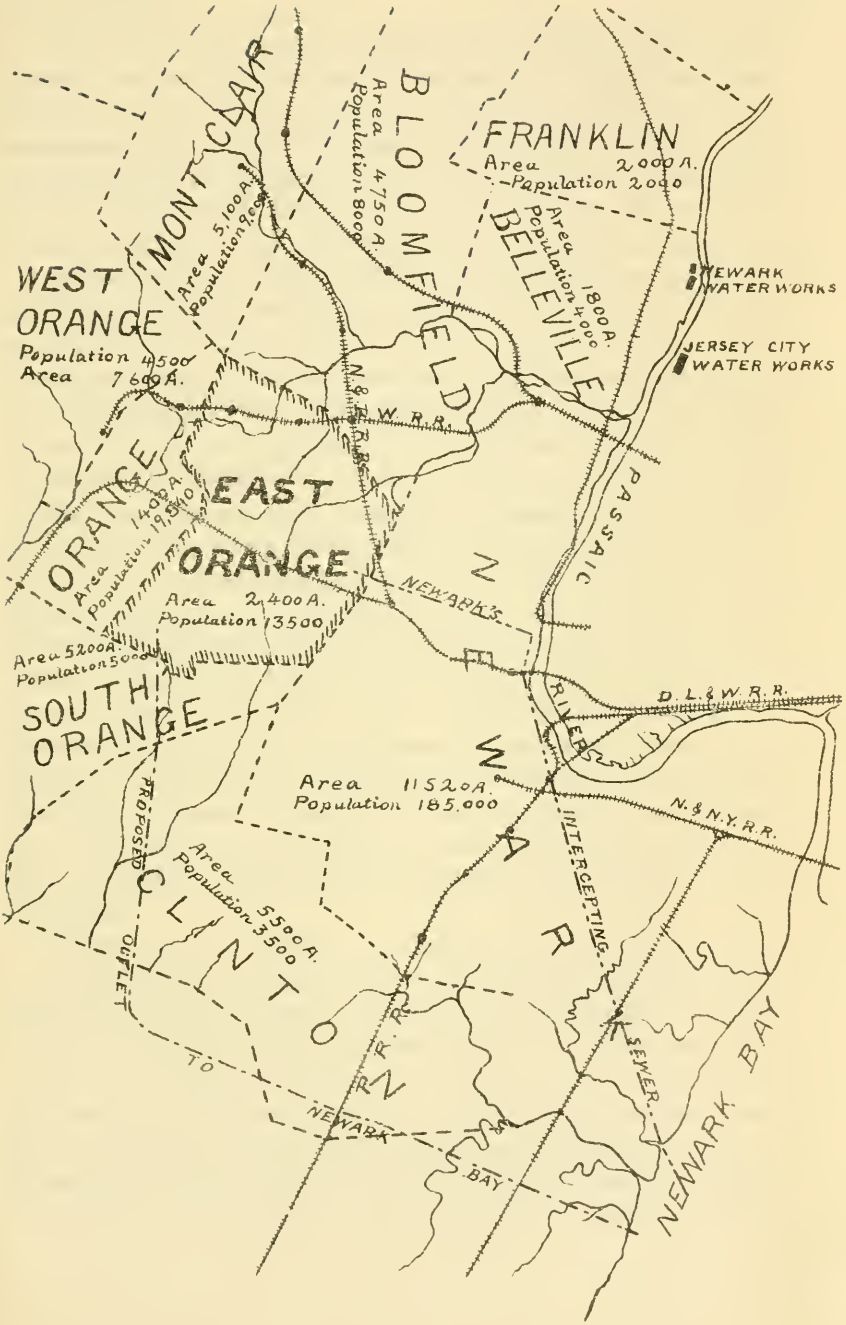

in circumference, the practical impossibility of making actually impervious sewers with vitrified pipe and cement under the conditions named, becomes apparent. But this flow from the 25 miles of pipe-sewers was limited to a small volume, probably about 2.5 gallons per second. It was necessary, however, to build the outfall sewer with a size beyond the maximum vitrified pipe, and a brick sewer was therefore constructed for 2 000 feet through a difficult formation, a timber cradle being used under the sewer. In another section of the town a tunnel was driven for about 2 000 feet at a depth varying from 25 to 35 feet, to avoid the interference with surface travel incident to so tedious a work in open cut. The great difficulty experienced in controlling the large volume of water encountered at this depth, practically prevented the construction of an impervious sewer in this place, where the tunnel was lined with brick. From these two pieces of brick sewer, less than a mile in length, about 5 gallons per second enters the sewers—twice the quantity, it will be noted, entering the remaining 25 miles of pipe-sewers. This aggregate flow of 7.5 gallons per second (650 000 gallons per day) mingles with the house sewage, becomes sewage and must undergo the purification processes. Such a flow is a material detriment to the operation of the purification works. Had information concerning the subsoil water of the township been secured—which the limited time available for the preparation of the plans prevented—underdrains to remove independently at least a large part of this flow would probably have been provided.

Plate XVIII is the general map of the township showing the sewer system as constructed.

On Plate XIX is shown the sizes and lengths of the sewers in the system.

The natural trend of the drainage of the township is to the north through four nearly parallel valleys, which break from the plateau forming the southern half of the township and dip to the north. In the northeastern district are located the water works supplying East Orange and Bloomfield. The water is here secured from shallow wells in the new red sandstone. It was expedient to keep collected sewage out of this district. It was determined to collect the sewage from approximately 2 000 acres, or five-sixths of the township area, by an intercepting sewer crossing the northern district from east to west (see Plate VII) into the Second River valley. The arguments in favor of this point of col-

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MAP OF

EAST ORANGE N. J.

SHOWING THE

SEWERAGE SYSTEM

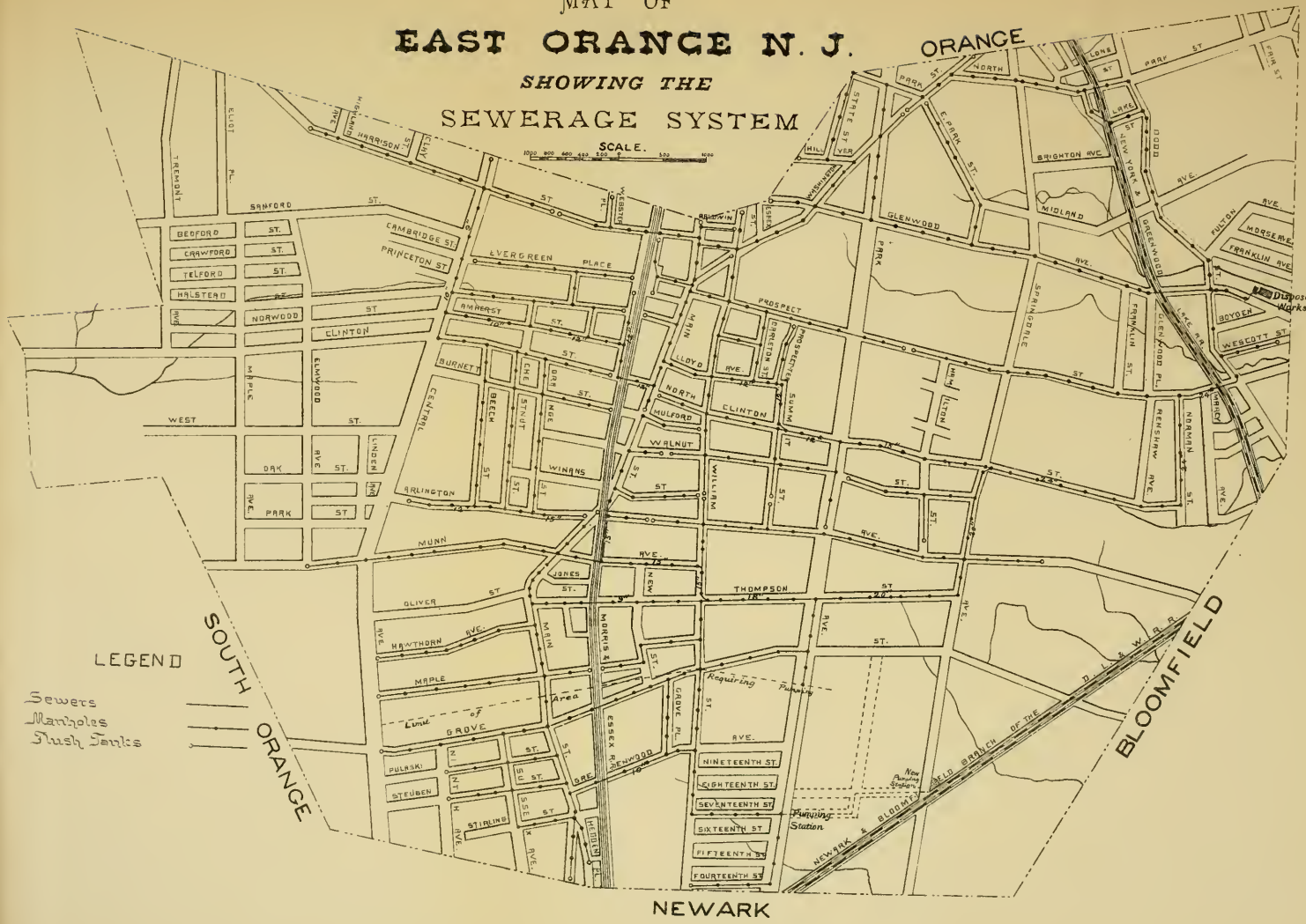

PLATE XX
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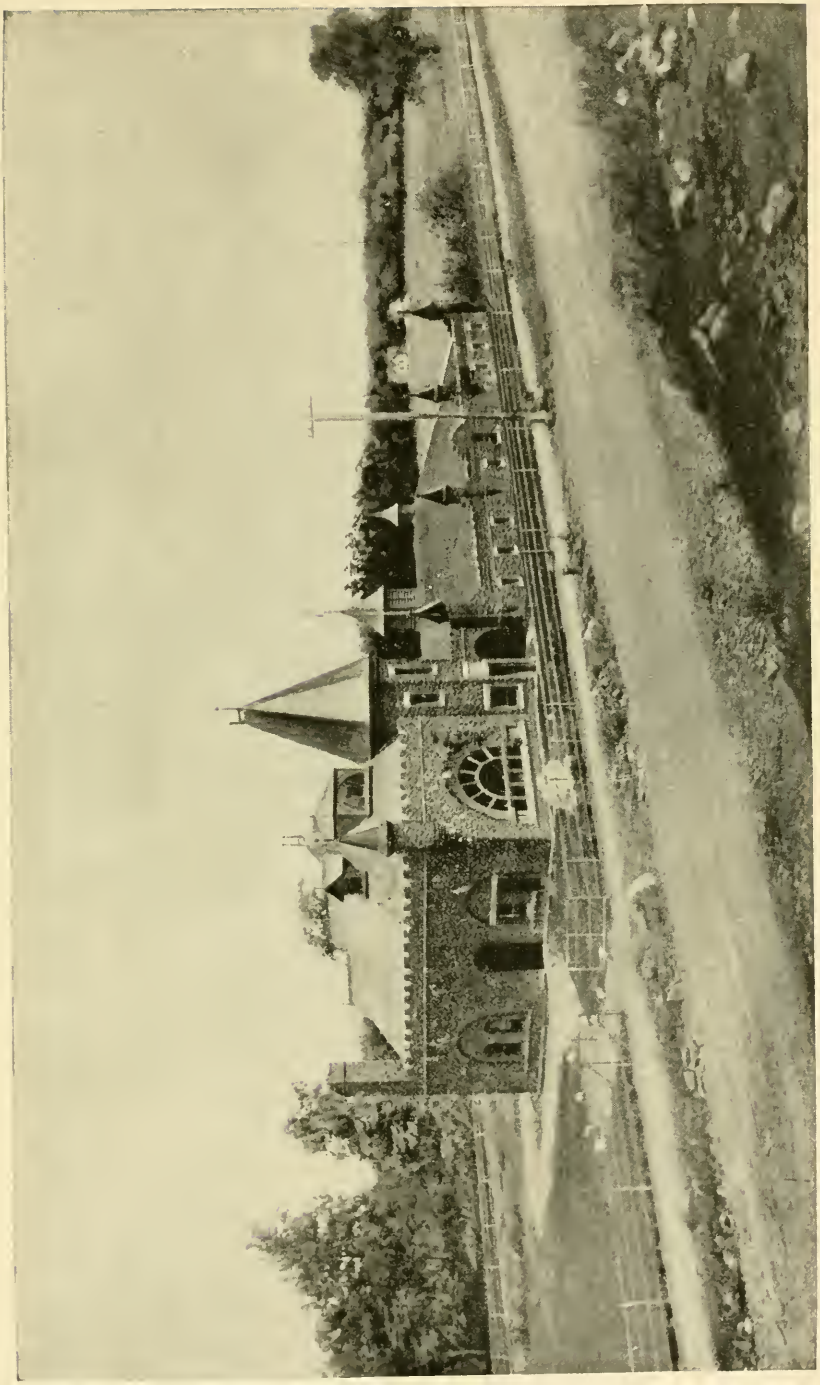

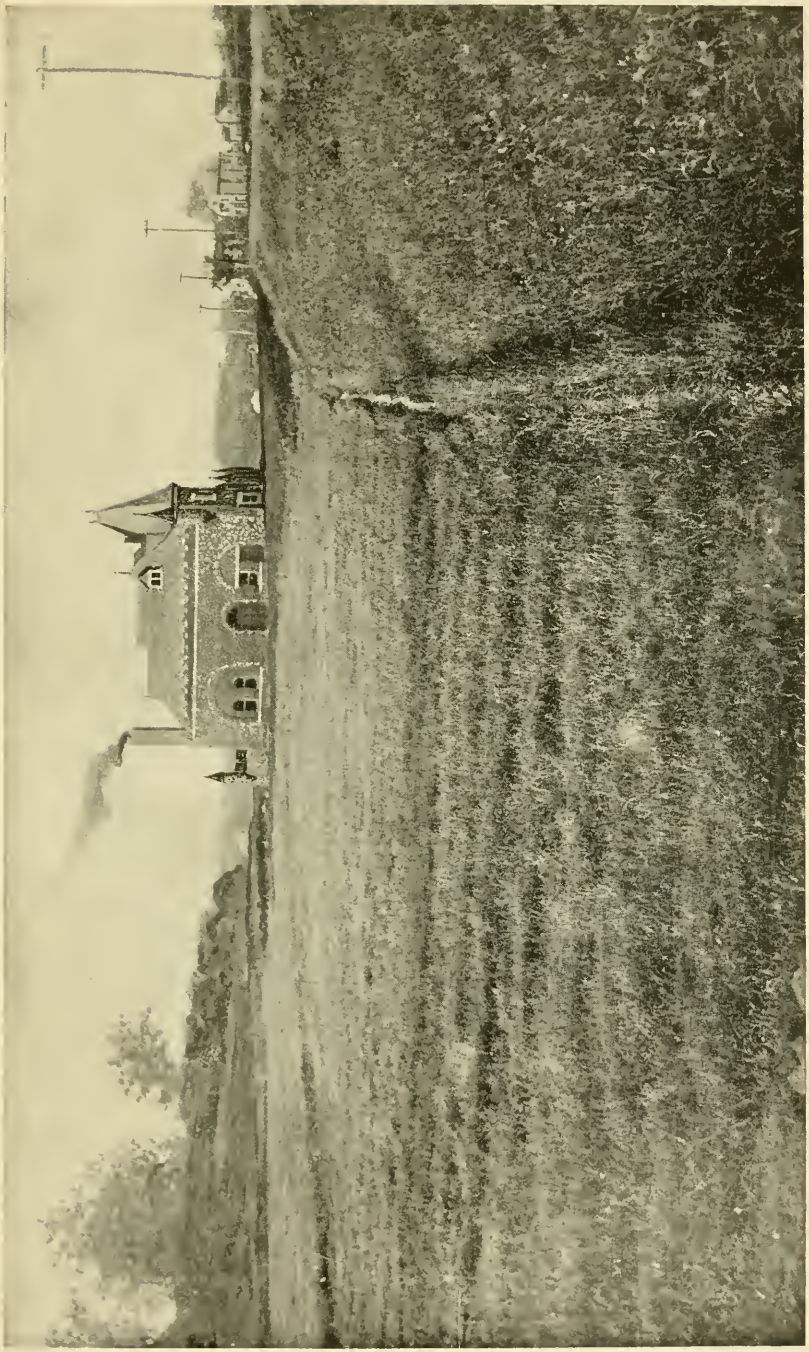

lection were: (*a.*) A larger percentage of the area of the township could be collected to this point by gravity, than to any other. (*b.*) The sewage would be united at the best point for ultimate gravity extension to tide-water, or combination with other towns if it were desired. (*c.*) About the only land in the township available for sewage treatment was here reached. (*d.*) The stream, offering outlet for the effluent, was larger than any other in the district.

The land secured for the works was singularly unfavorable for sewage purification. The total area available was about 15 acres; of this 5 acres were covered by Dodd's mill-pond, and the character of its bottom may be understood, when it is remembered that repeated complaint of its deposits had been made by residents to the Health authorities. The drainage and transformation of the pond was held out to hostile residents, as consolation for the location of sewage purification works in their midst. Reference is made to the general plan of the works, Plate XIX. This, together with the views shown in Plates XX and XXI, will show the residences immediately adjoining.

The stream indicated on the plan, originally fed the pond, but its channel has been deepened and straightened—a rather expensive piece of work, some of the excavation being made in quicksand. It is a tributary of the Passaic, called Second River. Its volume varies from 12 cubic feet per second, in dry weather, to 775 cubic feet per second flood volume. After a flow of about 4 miles it enters the Passaic, near the intakes for the water supply of Newark and Jersey City. Under these conditions it was necessary to secure a very high purity in any sewage effluent which was to be discharged into the stream, and the works must be operated without local nuisance. No reasonable expense was spared to make the works efficient and attractive; the buildings constituting the works are shown in the photographs Plates XX and XXI. A pleasing architectural effect is secured. The masonry is of high class; deep blue trap-rock, with rock face and worm joints, pointed with red mortar, relieved by red brick trimmings and cut stone capitals at the front about the doors and windows, secure a permanent and attractive appearance to the works.

The sewage enters the works in a 2-feet by 3-feet new form, egg-shaped, brick sewer; discharges into a conduit of rectangular section, having lateral projections extending nearly to its center on alternate sides at intervals of three feet along the axis. In this conduit, chemicals

from the building join the sewage; the lateral projections of the carrier give a whirling motion to the sewage, which causes a complete mixture of chemicals with it. The carrier leads the sewage to the precipitation tanks.

The tanks are constructed in duplicate, one set being cleaned or lying idle while the other is in use; Plate XXII gives a general plan of the building and tanks with longitudinal and cross sections. A brick wall located 10 feet in front of the inlet to the tanks, checks the velocity of entrance flow. A board floating on edge in vertical guides, intercepts the lighter floating matters, and insures their saturation before passing it. The cross-walls in each tank divide it into three compartments, and the flow passes over these with a depth of about 2 inches, the heavy matters being intercepted and settling. With a continuous flow of low velocity in the tanks, the surface water is being constantly skimmed off into the carrier. Drums float a swivel arm in each compartment, which connects with a low service pipe in the bottom of the tanks that discharges on the surface of the ground at a lower level. These arms draw water only from the surface, but the drums falling with the water enable any arm to empty the compartment in which it is located into the low-service carrier, leading to the surface of the grounds. The effluent from the precipation tanks, after entering the carriers (Plates XIX and XXIII), is distributed over the surface of the filtration grounds and descends to the under drains, which are from 3 to 5 feet deep and 20 feet apart over the entire 14.7 acres in the works.

The sewage effluent is applied to the land on the principle of intermittent downward filtration, the flow being applied successively to different areas. Part of the land is laid off in beds, 4-feet wide, separated by shallow furrows in which the water flows, and soaks laterally into the beds. The remainder of the land is divided into flat beds, 100 feet long by 50 to 100 in width, over the whole of which water flows. This latter method is preferable, as more water is disposed of, and in winter, frost is more easily kept out of the ground. Italian rye-grass has given the best results on the land, and is now grown almost exclusively. Farmers from the neighborhood cut the grass and remove it as is necessary, but up to the present time the town authorities have not been able to secure a satisfactory return on its sale.

Within the main building on the first floor are chemical mixing vats, filter presses, sludge-pressing machinery (receivers, air compressor and

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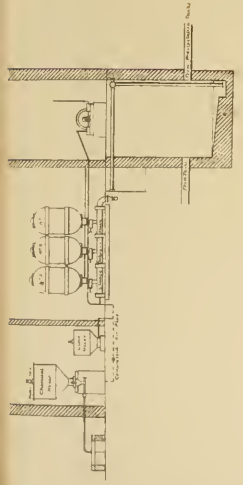

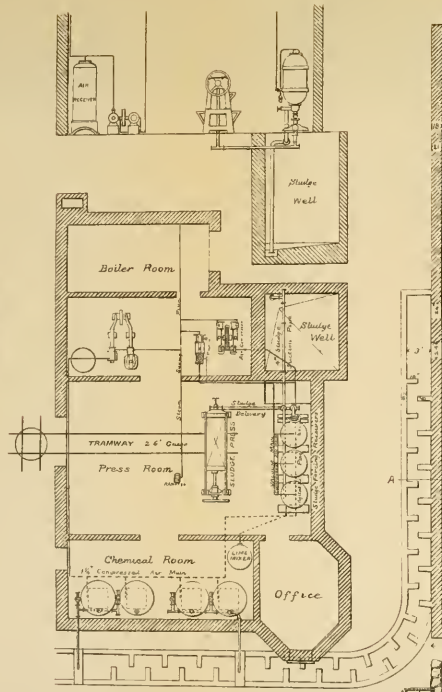

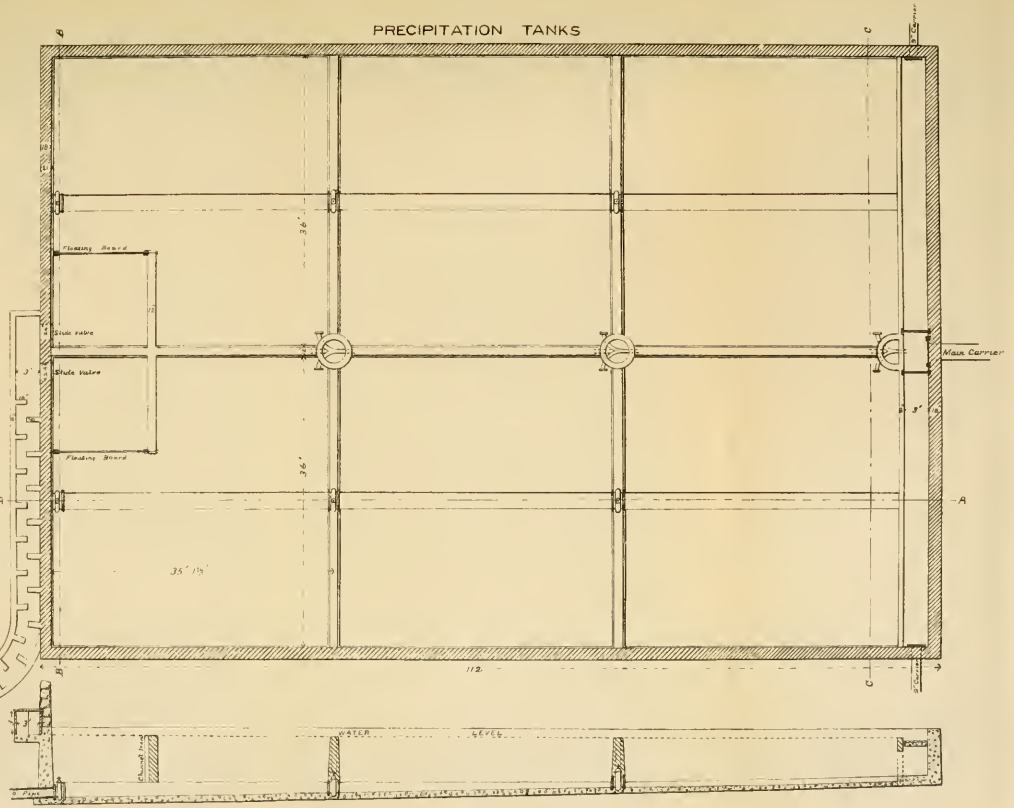

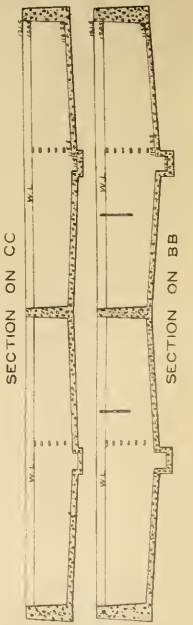

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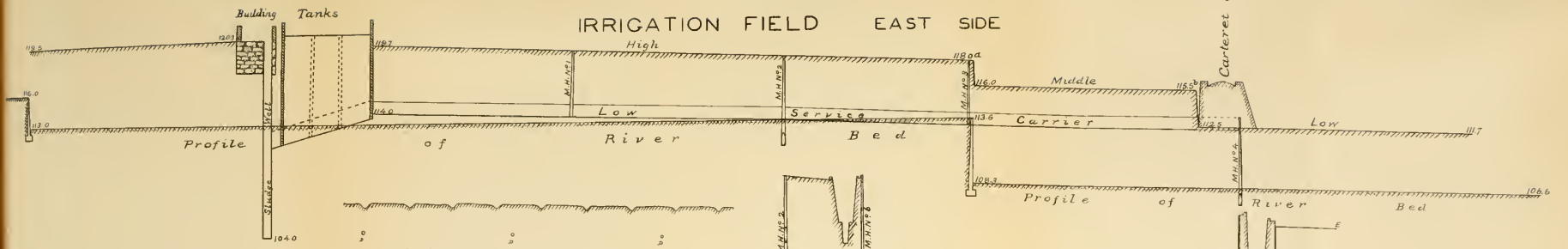
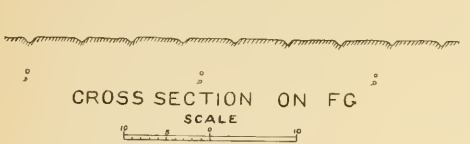
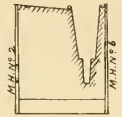
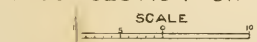
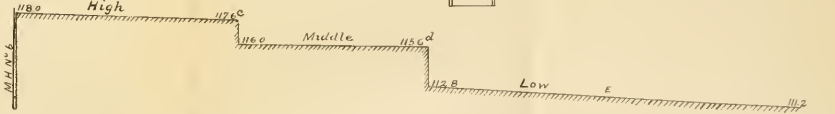
Profiles.











pump), boiler, and a small office for records and tests (Plate XXII). On the second floor chemicals and materials are stored. The chemical mixers are cylindrical cast-iron vats, 4 feet in diameter, with inverted cone-shaped bottoms overlayed with a perforated plate. The desired amount of chemicals is placed on the plate, water is let into the tank, and air blown up through the bottom, causing violent agitation of the liquid and resulting in the rapid solution of the chemicals. With a known flow of sewage at a given time, it is determined how wide to open a slide-valve in the bottom of the tank after solution of the chemicals is secured, in order to add the desired number of grains per gallon of sewage.

The sewage is mainly of a domestic character and somewhat constant in its alkalinity. Not more than 3 grains of lime and 2 grains of sulphate of alumina are now added to each gallon of sewage by the authorities, although when the works were placed in operation, the author recommended the use of 8 grains of lime and 10 grains of sulphate of alumina per gallon of the sewage. The present result is a less efficient precipitation. A combination of chemical precipitation and land filtration in the works makes it possible to increase the work performed by the land, by reducing the efficiency of chemical treatment, and *vice versa*. The labor of purification now placed upon the grounds is greater than its equitable share as originally intended. Much better results could be secured by calling out the full efficiency of the chemical treatment. To relieve the filtration grounds, which have rather a retentive soil; several artificial filter beds of coke and gravel were constructed under my direction, and have been of material service. (See Plate XIX.)

Returning now to the precipitated matter or sludge in the tanks; after the supernatant water is drawn off through the swivel-arm into the low-service carrier, a valve gate is opened and the sludge drawn into the deeper sludge-well within the building. By forming a vacuum in a cast-iron receiver, which is connected by an iron pipe with the sludge-well, the sludge is drawn up into the receiver, milk of lime being drawn in at the same time, by a small pipe from the mixing tank in the chemical room. This lime prepares the sludge for pressing, cutting the slime so that the water separates more readily from the solids. A pressure of 100 pounds per square inch is secured in one of the other receivers, and, being connected with the receiver containing the sludge by an air transfer main and the proper valves opened, the sludge is forced into a Johnson filter press and pressed into moist, hard, portable cakes.

An analysis of fresh sludge directly from the press made September 11th, 1889, by Mr. Charles T. Pomeroy, of Newark, N. J., gave results as follows:

Nitrogen from organic matter	.326 per cent.
Total phosphoric acid	.459 "
Moisture.....	50.625 "

Using the 1889 trade values adopted by the New Jersey experiment station we have an estimated worth of $1.51 per ton, 2 000 pounds. This sludge quite rapidly lost its moisture on exposure to the air, until it contained 6.37 per cent. moisture. If dried at 100 degrees C. it would have an estimated value of $3.06 per ton, 2 000 pounds.

The machinery used in manipulating the sludge was constructed by S. H. Johnson & Co., Stratford, England, who have erected numerous plants in England. Their machinery is the subject of several patents, and no similar devices are manufactured in this country. Not all of the machinery furnished has been satisfactory. The combination vacuum and compression pump and the high pressure water pump gave considerable trouble, and have been replaced by a Clayton compressor (5′ x 6′ x 7′) and a Worthington duplex pump (6′ x 4′ x 6′).

The method of maintaining and securing the pressure in the "hydropneumatic receivers" to press the sludge is worthy of brief comment. There are three receivers in the system (see Plate XXV); let them be represented by *A*, *B* and *C*. Air passes out at the top, sludge at the bottom, water enters at the side near the bottom and exits at the bottom. All the receivers are empty, except of air. Water is pumped into *A* and *B* and the air transfer mains opened to transfer their air to *C*, raising the pressure to 45 pounds per square inch. The valve on the air main from *C* is closed. The water in *A* and *B* is drained back into a shallow tank in the floor of the building with which the pump suction is connected—air taking its place. Water is again forced into *A* and *B*, operating proper valves, and the pressure in *C* is raised to 75 pounds per square inch. A repetition of this process brings the pressure in *C* to 105 pounds per square inch. *A* and *B* are emptied as before and a vacuum in *A* created, the sludge suction pipe is opened, and *A* is filled with sludge. Water is now pumped into *B*, forcing its air into *C* and thence into *A*, to force the sludge into the press. When *B* is filled with water, *C* may be filled with air, and *A* with sludge and air. When a receiver is filled with water, a float valve at the top closes the outlet to the air transfer

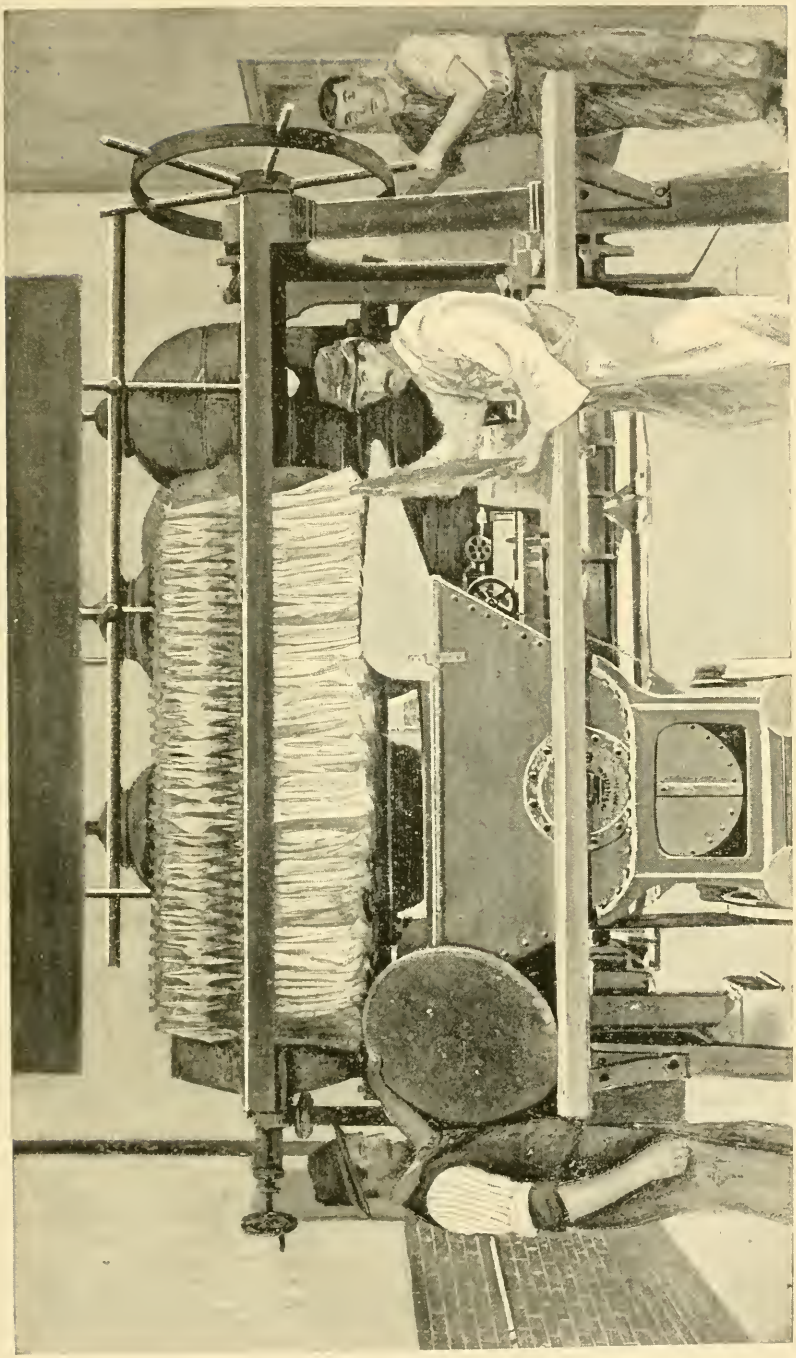


PLATE XXV
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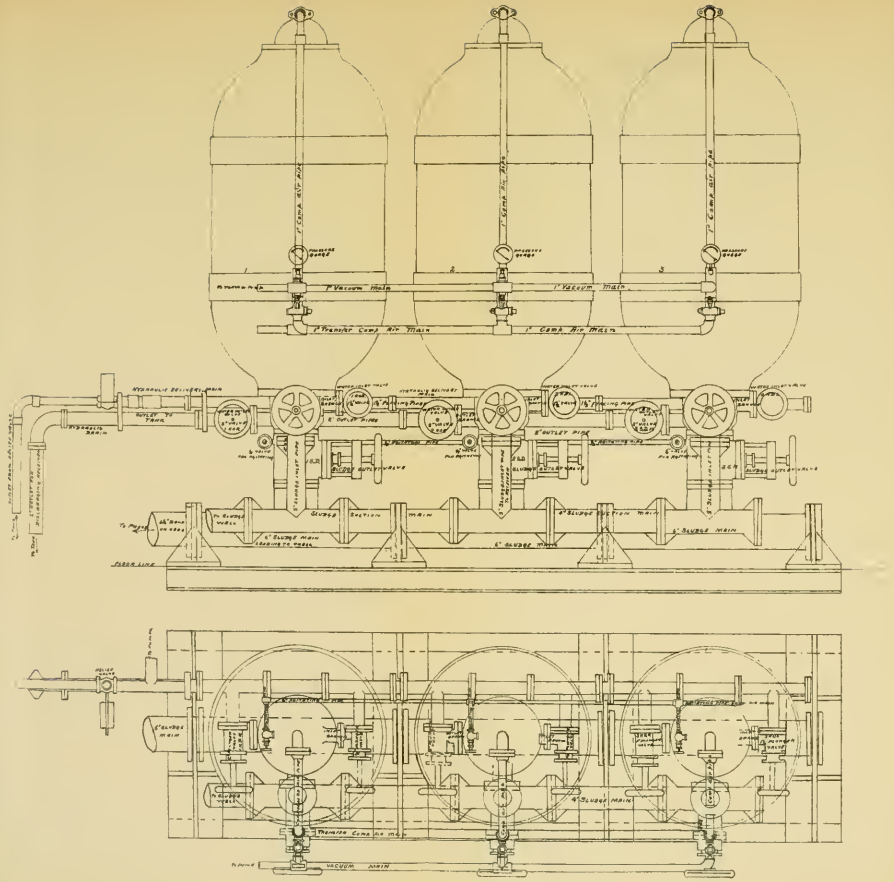
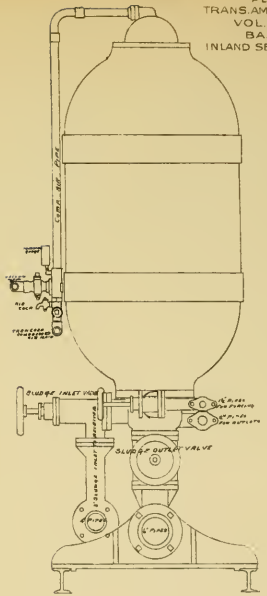
INLAND SEWAGE DISPOSAL.





ARRANGEMENT of SLUDGE FORCING RECEIVERS
FOR EAST ORANGE SEWAGE WORKS

C.P.H. BASSETT
DES'NG & CONS'T'G ENG.

main, and weighted valves on the force main lift and relieve the pressure on the pump.

The value of the process now first appears. *B* is emptied of water and filled with sludge, while at the same time water is being forced into *C* and by connection with *A*, forcing the remainder of its sludge into the press. Compressed air is thus never allowed to escape into the atmosphere. When *C* is filled with water, it is ready to be filled with sludge, while at the same time water is forced into *A* and the sludge of *B* forced in the press.

The filter press shown in Plate XXIV consists of thirty-six cast-iron cells, supported on a simple frame, with a central feed-passage into which the sludge is forced from the receivers. The cells are separated by canvas bags, and in the intercellular spaces the sludge remains, while the water is drained out through the canvas bags, into a trough on the rear of the press and returns to the tanks. On the end of the press is a capstan screw connected with a thrust block which presses the thirty-six cells of the press into close contact. It is the air pressure which separates the water from the sludge.

There is nothing offensive about these cakes when pressed dry; and if protected from water, after being taken from the press, they may be kept in bulk for weeks without nuisance. In the presence of heat and moisture they become more or less objectionable. The manurial value of the sludge cakes is slight. The small amount of precipitants used fails to retain the bulk of fertilizing matters in the sewage. At present between 8000 and 10 000 people are contributing to the sewage and about 13 tons of sludge are taken out each week. Some of the sludge cake has been sold at fifty cents per load; but more has been given away among neighboring farmers, while a large amount has been carted away by the authorities for burial, when no other removal offered.

A committee of the "Town Improvement Society of East Orange," an association of more or less intelligent gentlemen, determined during the summer and fall of 1890 to investigate the operation of the works and their results. They secured and submitted to Professor A. R. Leeds, two samples of effluent water for examination and report. The results generally are better than results secured for the author's use from time to time, but Professor Leeds' report is given in full as published by the Committee, as independent testimony.

Hoboken, N. J., September 29th, 1890.

Certificate of Water Analysis.

Sample No. 1. Taken from the flow as it emerges from the coke filter as the sewage leaves the tank.

(*a.*) From whom received, The Committee. No. 1447. When received, Sept. 8. Title of label, Sample No. 1. Source of Sample, E. Orange Disposal Works. Color, Turbid with white flocks. Taste, Not tried. Smell, Unpleasant, musty.

	Parts in 100 000.	Grains per gallon.
Free ammonia	0.087	0.051
Albuminoid ammonia	0.027	0.016
Oxygen reqired to oxidize organic matter	0.44	0.26
Nitrites	.00	.00
Nitrates	0.26	0.15
Chlorine	6.12	3.56
Total hardness	15.60	9.02
Permanent hardness	3.00	1.75
Temporary hardness	12.60	7.27
Total solids	29.60	17.26
Mineral matter	23.50	13.70
Organic and volatile matter	6.10	3.56

Other data, when required for judgment.

Interpretation of Results.

The results above stated indicate the presence of a large amount of unoxidized sewage. I should also suspect that this sample represents sewer water which has received the addition of some lime, taken at a point before the benefit of this treatment has been obtained. The effect of the addition of lime is to increase the temporary hardness (in the above analysis it is 12.6 parts per 100 000), after this lime has operated, by combining with the large amounts of carbolic acid contained in sewage, and the carbonate of lime resulting from this combination has precipitated, carrying with it much of the organic matter.

(Signed) Albert R. Leeds, Ph.D.,

Professor of Chemistry, Stevens Institute of Technology.

Note.—The United States gallon is taken at 58 318 grains.

Hoboken, N. J., September 29th, 1890.

Certificate of Water Analysis.

Sample No. 2. From the final effluent into the brook.

From whom received, The Committee. No. 1 448. When received, Septempter 8th. Title of label, Sample No. 2. Source of Sample, E. Orange Dis. Works. Color, White, clear. Taste, Not tasted. Smell, Not pleasant, slightly musty.

	Parts in 100 000.	Grains per gallon.
Free ammonia	0.02	0.012
Albuminoid ammonia	0.003	0.0017
Oxygen required to oxidize organic matter	0.40	0.233
Nitrites	.00	.00
Nitrates	0.38	0.22
Chlorine	4.00	2.33
Total hardness	20.00	11.63
Permanent hardness	12.50	7.23
Temporary hardness	8.50	4.40
Total solids	25.50	14.91
Mineral matter	22.00	12.83
Organic and volatile matter	3.50	2.08

Other data, when required for judgment.

It will be seen that the temporary hardness diminishes, and the benefit of the treatment becomes apparent. The second sample is strikingly different from No. 1. It shows an almost entire disappearance of the nitrogenous organic matter, the free ammonia being only $\frac{1}{40}$th and the albuminoid ammonia $\frac{1}{10}$th of the amounts present in sample No. 1. This disappearance is evidently due to oxidation, since the nitrates (which arise from the absorption of oxygen under the influence of nitrifying bacteria in the ground) are strikingly increased. I should suspect this sample to represent aerated sewage water which has passed through the ground. In its passage, its sewage impurities have been effectually removed, and the substances remaining are such as are found in country streams in their natural unpolluted condition.

Signed, Albert R. Leeds, Ph.D.,
Professor of Chemistry, Stevens Institute of Technology.

Note.—The United States gallon is taken at 58 318 grains.

To make these results the more apparent, the following table is presented:

	Free Ammonia.	Albuminoid Ammonia.	Oxygen required to oxidize Organic Matter.	Nitrites.	Nitrates.	Chlorine.	Total Hardness.	Permanent Hardness.	Temporary Hardness.	Total Solids.	Mineral Matter.	Organic and Volatile Matter.
	1.0 to 1.5	.30 to .70	.....	.....	.....	5 to 10	.....	.....	8 to 12	40 to 108	18 to 91	7 to 22
Raw sewage..... Parts in 100 000....	0.087	0.027	0.44	0.0	0.26	6.12	15.60	3.00	12.60	29.60	23.50	6.10
Effluent from..... Parts in 100 000....	0.051	0.016	0.26	0.0	0.15	3.56	9.02	1.75	7.27	17.26	13.70	3.56
Coke filter..... Grains per Gallon ...	0.02	0.003	0.40	0.0	0.38	4.00	20.00	12.50	8.50	25.50	22.00	3.50
Final effluent .... Parts in 100 000....	0.012	0.0017	0.233	0.0	0.22	2.33	11.60	7.23	4.40	14.91	12.83	2.08
Final effluent .... Grains per Gallon....												

In a letter of later date explanatory of these analyses, Professor Leeds stated:

"I should certainly feel far less sense of danger in drinking the sewage effluent, as represented by the samples sent from the East Orange

Sewage Disposal Works, than in drinking the water of the Passaic River, as pumped at Belleville and supplied to the inhabitants of Jersey City. Your effluent (September 8th, 1890) contained 0.0017 grains of albuminoid ammonia per gallon; the Passaic River, at Belleville, on the date of my last analysis (June 14th, 1890) contained 0.01 grains per gallon. In other words, taking the albuminoid ammonia as the measure of sewage contamination, the Jersey City water contains six times more sewage than the effluent waters from your works.

"As chemist for the Jersey City Board of Public Works, from 1881 to 1886, I found very many samples of the Passaic water even worse than the above.

"I regard the performance of your works and the character of the effluent as satisfactory from both the practical and sanitary standpoints."

The total cost of the works to January 1, 1891 (including about 4 miles of extensions constructed since my connection with the work), is given as follows:

Chargeable against sewerage system (29 miles)..	$322 020 64
Disposal works plant.....	75 098 60
Disposal works land (including 4 acres not used)	20 749 20
Total.....	$417 868 44

The cost of operating the works has been—

July, 1888, to March, 1889.....	$562 00	per month.
March, 1889, to March, 1890.....	746 00	"
March, 1890, to January, 1891.....	881 00	"

The average daily flow of sewage reaching the disposal works is approximately 1 300 000 gallons, or an average of about 90 000 gallons per acre. The need of the coke filters is therefore apparent. This daily flow may be approximately divided as follows:

Ground water from 25 miles, constructed under my direction.....	550 000 gallons.
Ground water from 4 miles, constructed since June, 1888.....	100 000 "
Flush tank flow.....	30 000 "
House sewage flow.....	620 000 "

Summarizing the processes of purification operating on the sewage in its passage through the works, we find subsidence, coagulation and precipitation in the tanks, mechanical filtration, aeration and nitrification in the soil. These principles include all that were practically available at the time the works were designed. More efficient precipitation might be secured, better land and more of it would be greatly appreciated, better business methods about the works and the disposal of its products are to be hoped for, and yet, under all the conditions involved, the plant is a success. It is doing its work under unfavorable circumstances, and doing it to the satisfaction of the East Orange authorities and its neighbors.

DISCUSSION.

Mr. Rudolph Hering, Director Am. Soc. C. E. (to Mr. Bassett).—You gave the quantity of ground-water entering the sewers. Was it determined before any sewage was turned into the system?

Mr. Bassett.—My original determination was made before any sewage was turned in, by gauging the outflow sewer and also by noting the length of time required to fill the tanks whose capacity was known.

Mr. Hering.—I remember asking you to do this, because later it would have been impossible. The present cost of maintenance you say is over $10 000 a year.

Mr. Bassett.—It is so reported by the township authorities.

Mr. Hering.—And the population contributing sewage to the works is between eight and ten thousand, making the cost of treating the sewage over $1 per head per annum. This expense is greater than it ought to be for the process used. I believe the excess is attributable to the fact that a large quantity of ground-water must be taken care of by the tanks and the filtration areas, instead of being discharged directly into the Second River. What did you say was the proportion of sewage to ground-water?

Mr. Bassett.—Pretty nearly equally divided.

Mr. Hering.—That, I think, explains the large expense. When the work was projected it was not apparent that so much ground-water was to be encountered, and great care was taken during construction to keep it out of the sewers. In order to prevent the ground-water from entering them and therefore getting into the tanks, the only practicable remedy is a system of under-drains independent of the sewers. I am not as sanguine as some are, regarding the possibility of obtaining a high degree of purity by any method of precipitation at a reasonable expense. Experience has shown both here and in Europe that it is not practicable

to so purify sewage by any known chemicals that the effluent water will not again putrefy. The value of precipitation must be sought in the removal of a large portion of putrescible matter, particularly what is suspended in the sewage. This leaves the effluent in a condition to be purified by filtration through a smaller area of land, or by dilution in a smaller body of water than would otherwise be possible. In other words, where insufficient land of a suitable character (sand) is available for the filtration of a given amount of crude sewage, such as was the case in East Orange, or, where the stream is too small to properly dilute the crude sewage, as in Worcester, Mass.—there a prior sedimentation or precipitation of some kind, aids us in finally getting a high degree of purity. The complete purification of sewage seems to be accomplished only by bacterial action, which, in the presence of oxygen, converts the organic matter into inorganic compounds. Oxygen, warmth, and time are essentials in the process. Slow and intermittent filtration through sand, or dilution in water having a large quantity of oxygen in solution, therefore, accomplish a complete purification of sewage in due time.

Experiments upon the chemical precipitation of sewage were made by the Massachusetts State Board of Health in December, 1889,* with lime and with salts of aluminum and iron, in order to discover their comparative value for the purpose. It was found that a certain definite amount of lime gave as good and better results than either more or less, and that this amount was equal to that which neutralized the carbonic acid in the sewage, or nearly so. It was further found that the more alum, copperas, or sulphate were used the better was the result; and that, while alum and ferric sulphate usually require no lime to perfect the precipitation, copperas requires a definite amount to do so. Copperas alone produced no better results than simple sedimentation. To accomplish precipitation by it, it is necessary to have more than enough lime to combine with the excess of carbonic acid over the amount to form bicarbonates and to combine with the sulphuric acid of the copperas. The relative value of the above chemicals will be shown in the following table, assuming the percentage of albuminoid ammonia removed as representing the organic matter, the cost of the chemicals being the same in each case.

Soluble Organic Matter Removed in Addition to all Suspended Matter.

	Per Cent.
Aluminum sulphate, material costing thirty cents per inhabitant per annum	20
Lime, material costing thirty cents per inhabitant per annum	22
Lime and copperas, material costing thirty cents per inhabitant per annum	29
Ferric sulphate, material costing thirty cents per inhabitant per annum	32

* Part II on Water Supply and Sewerage, 1890.

From the above experiments it appears therefore that at the same cost, ferric sulphate gives the best, and alum sulphate the least good results; also that the use of lime and copperas together, in the proper proportion, gives better results than either lime or copperas alone. It was found by the same series of experiments that both aluminum and ferric sulphate gave about as good results without lime as with it. The precipitation would be a little more rapid but hardly enough so to compensate for the extra cost. As lime and aluminum sulphate are the materials used at East Orange, it would seem that the use of copperas instead of alum, or still better ferric sulphate alone, would at a reduced cost give equally good results as those now obtained. If, still further, the ground-water could be excluded, the annual cost of the purification would be very materially lessened.

Alfred P. Boller, Mem. Am. Soc. C. E.—Mr. Bassett having referred to a committee of the Town Improvement Society, of East Orange, from whose report he does them the honor to quote liberally, as chairman of that committee, it is proper for me to add a few words to his interesting paper. This committee was the outgrowth of very grave charges against the efficiency of the Disposal Works, which implied the utter failure of the precipitation process, and that of the filtration of the effluent; resulting in the practical discharge of the sewage matter into the brook, substantially as it came from the outlet sewer. The result of the committee's findings completely disposed of these charges, the improvement being largely due to the betterments and administration of Mr. James O'Neill, the Township engineer in charge. Previous to his administration much trouble had arisen from the inefficient English machinery (referred to by Mr. Bassett); the failure to have an independent pump to return the thin sewage water from the sludge-well back to the settling tanks, instead of trying to pass it through the receivers only adapted to care for the thick and pasty sludge; and the failure of the filtration grounds which did not filter, due to the retentive nature of the clay soil, with the drain pipes 4 to 6 feet under the surface. These difficulties have been overcome by putting in new compression and water pumps, as stated by Mr. Bassett, and the addition of a pulsometer pump, to relieve the sludge-well of excessive water, and the introduction of coke filters in the form of beds of solid coke, some 50 feet long, 6 feet wide and 5 feet deep. Such beds are independent of all weather, but must be renewed when clogged.

To carry the sewage over the ridge from the low ground in the eastern portion of the township, an automatic gas engine had been located in a vault under Park avenue, near Grove street, which quickly proved to be utterly incapable of performing the work required of it. Much trouble, as can be imagined, grew out of this failure, but a new and efficient pumping plant, differently located, has been established and in

operation since July, 1890, correcting the evils this district suffered from.

The extent of leakage from ground-water (computed at about one-half the amount of sewage water treated) was obtained from gaugings at different times and hours in the outlet sewer as it enters the disposal works, compared with the number of times in twenty-four hours the settling tanks were filled. From this was deducted the legitimate work of the sewer system on the basis of the number of house connections and the water furnished by the Orange Water Company, the difference being ground water as near as it is practicable to obtain it. As Mr. Bassett truly says, the character of the excavations rendered tight work with vitrified pipes and cement joints absolutely impossible, and it is purely a question of individual judgment whether the leakage is in excess of what it fairly should be.

As the works are now operated, they can be called a success, and I cannot close my comments better than by quoting the opinion of the Committee at the close of their report: "The Committee are of the opinion that ultimately it will be found economical to dispose of the sewage by gravity to tide-water, which may become practicable through united action with the adjacent towns. The maintenance of a gravity system is practically nothing, while that of any local system, like that of East Orange, is a continuing and increasing annual charge. The town can well afford to capitalize such a charge and contribute accordingly to a gravity outlet, while the sewerage account could be reduced by the sale of the present filtration grounds. East Orange, however, can await with perfect equanimity the developments of the future, taking advantage of whatever may prove to be to its interests when the time arrives. Its present system of sewage disposal is a success and working satisfactorily, and can be made to serve the needs of a much larger population than the township now contains."

George W. Rafter, M. Am. Soc. C. E.—Mr. Bassett's paper is somewhat disappointing. Considering the lack of literature of sewage disposal in the publications of the Society his title led to the hope that the whole subject of sewage disposal in localities away from tide-water would receive a thorough overhauling at his hands. His preliminary discussion is, of course, in the line of such overhauling and will be of interest to those who have not given the subject special consideration. There are, however, a few debatable points raised, as for instance, Mr. Bassett expresses the belief that in some form of chemical precipitation lies the great hope for sewage purification in compactly populated districts. This may possibly be true, but in the present state of knowledge of sewage purification it may, I think, be considered doubtful, and I venture to submit a short discussion, not in the way indeed of attempting to prove absolutely the contrary proposition, but rather for the pur-

pose of showing that no sweeping rules of procedure, governing every case arising can be laid down.

In studying sewage purification historically it is found that the weight of opinion has vibrated pendulum-like between the different methods, and for the last few years chemical methods have been somewhat in the ascendant. In the case of the town of East Orange, it appears clearly necessary to manufacture potable water from the sewage, and the inadequacy of the chemical method by itself to do this, is indicated by the character of the design proposed by Mr. Bassett for the sewage purification works at that place. The chemical analyses of the effluents made by Dr. Leeds are of interest as showing the relative value of the chemical methods as compared with purification by land. On the question of relative cost I shall invite attention directly.

From Dr. Leeds' analyses it appears that at East Orange the effluent from the chemical purification process is still practically unpurified sewage. This is emphasized by the remark of Dr. Leeds, to the effect in his interpetation of results, that he suspects "that this sample represents sewer water which has received the addition of some lime, taken at a point before the benefit of this treatment has been obtained." This is especially significant as to the value of the chemical treatment when we consider that the sample on which Dr. Leeds makes this report was, according to the record, "taken from the flow as it emerges from the coke filter as the sewage leaves the tank." That is to say, if I understand the matter correctly from Mr. Bassett's written description, this sample was taken after the chemical treatment and after the sewage had further flowed through a coke filter. The second sample was from the final effluent after the land purification, and represents water, against the potable qualities of which no valid objection could be urged from the chemical point of view. Dr. Leeds, however, records the smell as "not pleasant, slightly musty," a condition possibly superinduced by the character of the soil through which the filtration takes place. Mr. Bassett remarks that these two analyses of effluents as made by Dr. Leeds are generally better than the results secured for his own use from time to time. If Mr. Bassett has anything like a complete series of analyses of these effluents it would be of considerable value, as assisting in settling this question of the relative efficiency of the chemical and land purification methods, if he would introduce them in his final discussion.

It is now well understood that a single series of analyses cannot be considered as indicating anything more than the condition of things at the time of taking, but if those made by Dr. Leeds are fair averages, we may conclude that the analyses now before us are a striking illustration of the truth of some remarks on the insufficiency of chemical methods made by Dr. Carl Pfeiffer of Wiesbaden in 1888, namely:

"For some incomprehensible reason, this entirely impracticable and,

as regards its results, most unsatisfactory mode of treatment has, during the last few years, grown much in favor, and the author is of the opinion that the time has arrived when a strenuous opposition should be offered to these so-called clarification processes, and, when in the interests of municipal authorities, a warning should go forth against the excessive cost of the chemical systems of treatment as compared with the good they can effect."

Where conditions similar to those at East Orange exist, where it is absolutely necessary that the purified effluent pass into streams used for water supply below the sewage outfall, the prime object of sewage purification is and must always continue to be the removal not only of suspended and dissolved organic substances but, what is of far greater importance, of disease germs. The analyses submitted by Mr. Bassett show that the chemical treatment had removed only a portion of the organic matter, and we have absolutely no guarantee under these circumstances that any disease germs have been removed at all. Probably, however, the coagulation and attendant precipitation superinduced by the addition of the chemical reagents may have carried down into the sludge some of the bacterial germs of disease if any were present, but that anything like all the germs are ever removed in this way we have as yet no reason for assuming.

In view of the foregoing, let us examine (1) as to whether the work actually accomplished by the chemical treatment is really worth what it costs; and (2) whether in the light of present information the same work could not be better accomplished by other methods.

Mr. Bassett states that at the present time lime is added at the rate of not more than 3 grains per gallon of sewage and sulphate of alumina at the rate of 2 grains per gallon, and that the daily flow of sewage is at the rate of approximately 1 300 000 gallons. For purposes of the discussion I will assume that lime of the proper quality is delivered at the works in the bins ready for use at $8 per net ton, and sulphate of alumina at $22 per net ton. The yearly cost for these chemicals becomes then:

Lime, 101.6 net tons, at $8	$812 80
Sulphate alumina, 67.8 net tons, at $22	1 471 60
Annual cost of chemicals	$2 284 40

With the assumed prices the foregoing represents the annual cost of chemicals under the present system of operation, while the total cost of operation is now, as per statement, $881 per month or $10 572 per year. It would assist greatly in studying the question of efficiency of these disposal works, if Mr. Bassett would give in the final discussion the detail, as for instance, cost of fuel, oil and waste, chemicals, attendance for the chemical treatment purely, the same for the land purification process, together with the cost of renewals and repairs for each.

The cost of operating the Johnson Filter Press would also be an interesting and valuable item of information, especially since this is, so far as I know, the only Johnson press in operation in this country. As the matter now stands the cost of the operation of those works appears to be more than $1 per head per annum for the population actully served, and this must in our present light be considered somewhat high, even in a case where the conditions require the manufacturing of potable water.

Taking my assumed annual cost of chemicals, it appears that the other items of expense are as a whole ($10 572 00 — $2 284 40) = $8 287 60. Mr. Bassett, however, tells us that his original recommendation was 8 grains of lime and 10 grains of sulphate of alumina per gallon of sewage. Let us consider the total annual expense if this recommendation should be carried out:

Lime, 271.2 net tons, at $8.....	= $2 168 80
Sulphate alumina, 338.9 net tons, at $22.....	= 7 455 80
Annual cost of chemicals.....	$9 625 60

Adding this amount to the actual cost at the present time, exclusive of chemicals, and we have ($9 624 60 + $8 287 60) = $17 912 20, a sum representing approximately $1 80 per head of population per annum.

Mr. Bassett is apparently under the impression that the greater the use of chemicals the greater will be the degree of purification attained by chemical treatment, but experience shows that beyond a certain limit no material advantage accrues from the use of additional chemicals. This conclusion is supported by the result of trials at the Mystic Valley Sewage Purification Works as detailed by Wilbur F. Learned, Member of the Boston Society of Civil Engineers, in a paper read February 15th, 1888; from which it appears that the addition of sulphate of alumina at the rate of one-half ton per 1 000 000 gallons precipitates at the Mystic Valley Works 25 per cent. of the total matter of the sewage, while two tons per 1 000 000 gallons precipitates generally only abont 30 to 32 per cent. of the total matter. In this case but one reagent is used, and it is only fair to state that better results are obtained by the use of two reagents than one. My only object in citing this experience at the Mystic Valley Works is to illustrate the principle that beyond certain limits the addition of chemicals has little effect in the way of assisting the purification. Moreover, it may be noted that the amount of chemical reagent to be used will depend to some extent upon the amount of dilution of the sewage. Dr. Lidy states that for a successful lime treatment the quantity of lime added should be at least 10 grains per gallon, to a sewage that does not exceed 30 gallons per head of the population, but probably this amount was intended for sewage, the watery

portion of which is comparatively hard. Where two reagents are used the amount of each is much less than with only one, and for the lime and sulphate of alumina process 5 grains of each may be taken as a maximum for sewage of medium strength.

The precipitating effect of lime on sewage is due to two causes: (1) the combination of some of the lime with free and partially combined carbon dioxide, forming an insoluble carbonate of lime; and (2) to a further combination of an additional part of the lime with a portion of the organic matters in solution. The insoluble substances so formed subside, carrying with them the major portion of the suspended matters in the sewage, and sinking to the bottom they form sludge.

Sulphate of alumina exercises a precipitating effect by virtue of a combination of the sulphuric acid with lime and other bases in the sewage, whilst alumina hydrate forming a flocculent precipitate entangles and carries down the suspended organic matters. The most of the authorities now recommend the lime and sulphate of alumina treatment, the proportions in which they are used being such as to yield as nearly as possible a neutral effluent. It is evident, therefore, that no hard and fast rule as to the amount of reagent to be added can be laid down. Theoretically, frequent tests should be made of the nature of the sewage as delivered at the disposal works, and the chemical treatment adapted to the varying conditions of the flow. Practically, also, this has been found to be the best method of procedure, and it is stated that at Worcester such tests are made, and the application of chemicals gauged in accordance with the results thereof, as often as once every half hour.

At East Orange, with a population of 10 000, and a flow of sewage of 1 300 000 gallons in twenty-four hours, we have a daily flow per capita of 130 gallons. With such a dilution of the sewage, I question somewhat Mr. Bassett's conclusion, that very material improvement of the effluent from the chemical treatment will be obtained by the use of the quantities originally recommended, over what is now obtained. Certainly the slight gain in purity of effluent would not be commensurate with the expense. My reasons for this opinion I indicate in the foregoing. There are, moreover, good grounds for believing that the purification of the quantity of sewage treated at East Orange could be efficiently accomplished, even on the limited area there available, by the process of intermittent filtration, at materially less expense. The experiments of the Massachusetts State Board of Health have pointed out to us the rational method of procedure, and much of the information available upon the subject of intermittent filtration when Mr. Bassett designed these works is obsolete, by reason of the new views which we derive from the Massachusetts experiments. I will not take time in discussing at length the results which have been obtained in these experiments. The large volume containing the record of them is now in process of distribution, and it is sufficient for present purposes to say, that with a

sewage as dilute as that at East Orange, there is no reason to doubt but that intermittent filtration areas can be constructed, capable of taking care of and efficiently purifying for all time to come, from 75 000 to 100 000 gallons of sewage per acre per day; and taking 75 000 gallons per acre per day as the basis of an estimate, we find that 17 acres would be the area required. I understand, however, that 19 acres were actually purchased at East Orange, and assuming that this is a nearly level tract, we will, for purposes of comparison, estimate the cost of preparing this area for high grade intermittent filtration, in accordance with the Massachusetts views. In this estimate I take the price of sand at $1 per cubic yard, in place in the work, the assumption being that for the quantity required to make a depth of 3 feet over the whole area, this price would be ample, even under somewhat unfavorable conditions, as for instance, long haul and the necessity for laying some temporary track, in order to transport sand from the bank by railway transportation.

A filtration field of this character will be prepared, under the conditions which obtain at East Orange, where the sewage can be easily delivered upon it, with no other assistance than gravity, by first leveling the natural surface in beds, substantially as has been actually done by Mr. Bassett. After such leveling the under drains will be laid, the trenches refilled, and selected sand of the proper quality filled in upon the previously leveled and prepared surface, the material moved in the process of grading and leveling having been used to form the necessary embankments at the sides. The designs of the distribution carriers and necessary arrangements for straining out coarse material, is a matter of simple detail, which need not be considered here.

The estimated cost of such a field, sufficient for the work to be done at the present time at East Orange, may be taken as a matter of illustration, subject to the limitations indicated, as follows:

19 acres of land, as per paper		$20 749 20
19 acres leveled and graded, at $300		5 700 00
19 acres drained, at $250		4 750 00
19 acres furnished with sand 3 feet deep, at $4 800		91 200 00
Distribution carriers and arrangements for straining out coarse material		10 000 00
Barns, sheds, teams, wagons, tools, etc		2 000 00
Contingent expense		15 600 80
Amount		$150 000 00
Annual cost of operation :		
1 foreman, at $75 per month	$900 00	
6 laborers, at $35 per month	2 520 00	
Keeping teams, repairs of tools, etc	1 000 00	
Amount	$4 420 00	
$4 420 capitalized at 4 per cent		110 500 00
Total capitalization		$260 500 00

Taking the cost of the purification works as actually constructed, as stated by Mr. Bassett, and we have the following:

Disposal works, plant	$75 098 60
Disposal works, land	20 749 20
Total cost of disposal works	$95 847 80
$10 572, the present cost of operation, capitalized at 4 per cent., gives	264 300 00
Total capitalization	$360 147 80

From which it appears that something like $100 000 capitalized investment would be saved at East Orange by the method of intermittent filtration, the comparison being made with the actual cost of the chemical process as carried on at the present time. If, however, we assume for the sake of the argument, that the chemical treatment originally recommended be applied, we find that the difference in the total capitalization in favor of high grade intermittent filtration will be $283 152 80.

I have gone somewhat more into the detail of this matter than I intended when I started out, and the only excuse I can offer is that it is a subject of absorbing interest to those who have studied it sufficiently to become familiar with the detail. I hope the introduction of such detail in the discussion is not wearisome to the Society. There is, however, one other point growing out of the Massachusetts views as to intermittent filtration, namely, that the preparation of these high grade filter areas in localities where the proper material is available, by reason of reducing very materially the number of acres required to filter the sewage of a given population, has a most marked effect upon the cost of maintaining and operating the intermittent filtration areas. Obviously much less labor will be required to operate 20 acres than would be required for 60 acres, and we therefore must conclude that the foreign experience in the cost of operating intermittent filtration areas can have in the present understanding of things, relative value only. The use of such data, without modification, will lead to erroneous results, and estimates of the cost of operation which do not take into account the new views are behind the times.

Enough has been said in the foregoing to indicate that there is a good deal of doubt as to chemical treatment being the great hope for sewage purification in compactly populated districts. Indeed, where a high degree of purity of effluent is required it can be usually attained at less cost by intermittent filtration alone than it can be by the combination treatment which Mr. Bassett has used at East Orange. At the same time, I do not wish to be understood as pinning my faith exclusively to intermittent filtration. In sewage purification, of all things, the engineer must look at the question from every side.

Mr. Bassett's closing remark that better business methods about the works are to be hoped for, leads to the inference that he has probably experienced in connection with this design some of the difficulties surrounding the engineer striving to attain an ideal in the prosecution of municipal work. Just how difficult this is, under the present universal system of political management, is well understood by all honorable municipal engineers. Such criticism as is indulged in as to the apparently high cost of operation of these works is fully subject to modification, due to the probable political co-efficient, and I trust that Mr. Bassett will understand, therefore, that my criticisms are made merely in the hope of eliciting the truth.

C. Ph. Bassett, M. Am. Soc. C. E.—In closing the discussion I desire to state that I have had no official connection with the East Orange works since their completion and delivery to the town in the summer of 1888. Management of the plant has not been under my control, and my information regarding expenses is necessarily obtained from officials or town records. Many of the expenses chargeable against the works are properly chargeable to betterments and experiments, and not to maintenance, but I have been unable to fully separate these items. This is seriously regretted, for a much more gratifying showing could be made were items of maintenance proper placed alone.*

The remarks of Mr. Boller apparently convey the impression that the coke filters at the works are recent additions. They were in fact built originally under my direction in 1886 and 1887. They have, however, been renewed. It might also appear from the temper of Mr. Boller's reference to the small pumping station in the eastern section of the town, that some censure was due the original location and design. The station was constructed as a vault beneath Park avenue (as indicated on Plate VII), because the territory to the north was entirely undeveloped and could not be opened by an outfall sewer without troublesome litigation. The committee in charge accepted the location as an expedient until the property was opened at the will of the owners. Steam could not be used in a vault beneath this important throughfare, and a

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* I have ascertained from Superintendent O'Neill that the expenses of operation at the disposal works at the present time are approximately as follows:

Engineer and laborers at building, coal and water, oil and waste	$300	per month.
Chemicals, including lime	200	"
Manager and two helpers on grounds	155	"
Removal of sludge	70	"
Total	$725	"

There are probably 15 000 people contributing to the sewage at the present time. The annual per capita cost of maintenance, therefore, exclusive of interest charges is about 60 cents.

It is probable that this *per capita* cost will reduce as experience and sewage volume increase.

Baldwin gas engine geared to two small Worthington pumps was used. The phenomenal growth of the district served by this machinery, immediately following the construction of the sewers, made its usefulness shortlived; but the growth caused streets to be opened in the lower district, and made possible the construction of the pumping station practically where originally located by me.

Kindly remember that the works described in this article were designed in 1886, and my original paper was presented at the Cresson Convention in June, 1890, and now appears substantially as then read. The works were the first in this country to apply a systematic chemical treatment of sewage. European experience alone guided their design. The preliminary statements of the paper were intended to outline what had then been done in this country in sewage purification. Since June, 1890, much work of experimenting and actual construction has been accomplished. Members in their discussion have had access to this recent information, and naturally, have been surprised that it did not appear in my paper now published. I personally much regret that the paper did not appear soon after it was first presented. It may be mentioned that the delay was not entirely mine. It is not my intention in this discussion to bring the subject fully up to date; that must be left for later and more comprehensive papers. But I desire to add a few words along the lines originally considered, and reply to the inferences drawn by members in their discussion from recently acquired information which reflect on the principles of the works as designed in 1886.

It is to be regretted that provision for underdrainage in parts of the East Orange system were not made. The conditions conspiring against this result have been outlined, but it appears desirable to emphasize a statement in my original paper, that the large ground water flow entered the system mainly from about 3 200 feet of brick sewer. At the outlet sewer (1 700 feet long) ground water was not expected in large volumes, and the work began in a dry trench; an under drain was not therefore laid. This work later penetrated a very wet quicksand. At the other point (1 500 foot tunnel), objection was raised by the committee in charge to the expense of underdrainage into a lateral valley. Had the water been excluded from these two pieces of brick work, the ground water flow from the 25 miles of sewer would not have exceeded 400 000 gallons per day. The expense of operating the works would then have been much reduced. The quantity of ground water flow as I have given it in the original paper was ascertained by me before any house connections were made. Mr. Boller's remarks on this matter I fancy refer to the calculations of the committee which appear to accord well with my own gaugings. For the sewerage of Orange, N. J., immediately adjoining East Orange, and in the same class of construction, I am at the

present time securing effective underdrainage in what is called "liver" and running sand, and securing practically dry sewers.

I am at one with Mr. Hering in his statements regarding the present limitations of chemical precipitation, with one exception. He appears to me to underestimate the value of this process. He states that the experience of Europe and this country has shown that it is not possible with any known chemicals to so purify sewage that it will not again putrefy. Recent reports, now in hand from Europe, are not entirely in harmony with this statement. Sir Henry Roscoe, in his report on the effluent from the Webster electrical process, states: "I have not observed in any of the unfiltered effluents from this process, which I have examined, any signs of putrefaction, but on the contrary, a tendency to oxidize." Alfred E. Fletcher, Esq., F.C.S., Inspector under the Rivers Pollution Prevention Act in Scotland, states: "The results of my examination of this process convince me of its efficiency in preventing putrefaction of the effluent." J. Carter Ball, F.I.C., etc., states in his report on the experiments at Salford, England: "In comparing the two systems the electrical and the polarite, it is rather difficult to state which yields the better effluent in regard to liability to putrefaction. Many of the effluents have been kept from both processes for weeks and months in a warm place in open vessels, and at the end of that time there was not the faintest odor of decomposition, and in most cases the water is clear." Prof. E. Frankland, D.C.L., comments in a similar vein upon the process at Acton, England.

I have entered thus fully on this matter since the expression of my belief that in some form of "chemical precipitation lies the great hope of sewage disposal in compactly populated districts" has occasioned some criticism. This belief did not (in 1890) grow out of the superior results then obtained from chemical processes, but from a consciousness of the deep-seated popular aversion to a sewage farm near habitations, and a realization that in many districts, owing to lack of unoccupied land, land disposal is a practical impossibility. If chemical processes can be sufficiently perfected so that a large percentage of the dissolved—and practically all the suspended matters—can be economically removed, we will see less effort made to unite large thickly settled areas in gigantic projects for conducting sewage to distant tidal waters or large streams. The perfection of land processes alone will not secure such results. I have expressed the hope that chemical processes will be thus perfected.

With Mr. Rafter's position I must squarely take issue. He apparently concludes that a land treatment would have been preferable to the dual system recommended and placed in operation in East Orange. With the general principle that land treatment is better than any other process of purification, where the conditions are favorable, I am in accord. But in his application to East Orange he has, beyond question, fallen into the error against which he cautions, viz.: "That no sweeping rules of

procedure governing every case arising can be laid down." My professional work has included tidal discharge, mechanical straining, broad irrigation, land filtration, chemical treatment and combinations of these principles, varying with the conditions treated. The combination process constructed in East Orange was not the result of blind infatuation with any method, but was recommended because a painstaking examination of the local conditions appeared to justify no other course. It is fair to assume that Mr. Rafter is not familiar with the local conditions surrounding the works, and has not fully comprehended some of the difficulties with which they have grappled, otherwise it would be hard to account for some of his arguments or deductions. To demonstate moonlight to be cheaper as an illuminant than gas or electricity, does not thereby render it universally practicable. Because the Lawrence experiments show that filtration through clean gravel give better results than other processes attempted, does not thereby demonstrate the universal applicability of that process.

It can, in my judgment, be clearly shown that land treatment alone would not have been entertained, and would hardly have been practicable in East Orange, but such is hardly within the scope of this discussion. I realize, however, that Mr. Rafter's position and conclusions are so far from my own that they require some further comment. He assumes that my knowledge of chemical treatment is sufficiently limited to lead me to conclude that the purity of the effluent will increase uniformity with the chemicals! His reason for this assumption appears to be my statement that 3 grains of lime and 2 grains of sulphate of alumina per gallon of sewage, as at present added, are too small to indicate the results obtainable by chemical precipitation. He then cites Mr. Learned's experiments, where 7 grains of sulphate of alumina per gallon removed 25 per cent. of the total solids in the sewage, and 14 grains removed 30 to 32 per cent. These experiments (using chemicals in each case in excess of those in East Orange) appear to support my judgment, that more chemicals should properly be used to relieve the labor of the land.

Mr. Rafter's estimates of land preparation as compared with the chemical process are in no sense applicable to the conditions met in East Orange. He fails to consider the cost of lowering Second River through the grounds (about $16 000) as part of the land preparation. He overlooks the fact that a very expensive building—needlessly expensive so far as the effectiveness of the process is concerned—is chargeable to the precipitation plant as now constructed. He neglects to allow for the excavation of the grounds before the sand and gravel are added, as he proposes; this excavation would be needed over 7 acres to a depth of 3 feet, since the out-fall sewer (grade 1 per 1 000) could not be raised and is now level with the surface. If this excavation were not made before the gravel was added pumping would be required. He proposes to use 19 acres while only 14 are available, the remaining 5 being pur-

chased with the original tract to serve to separate the grounds from numerous houses in Bloomfield.

My judgment is that the principles involved in the East Orange works are those most competent to grapple with the difficulties there existing; but I am satisfied that better results may be obtained from the precipitation without increased expense by changing the chemical process.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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Vol. XXV.—August, 1891.

AMERICAN IRRIGATION ENGINEERING.

By HERBERT M. WILSON, M. Am. Soc. C. E.

WITH DISCUSSION.

The development of the scientific practice of irrigation engineering in this country is a thing of to-day. Until about 1882, there can scarcely be said to have been constructed a single irrigation work designed on sound engineering principles. The art of American irrigation engineering has only been developed within the past few years, while the majority of the more modern and creditable works are but now approaching completion.

The early works of Colorado and California were all hastily constructed under great pressure from their owners, in order to realize some moneyed return at the earliest possible moment, and as a consequence the charges for maintenance and repairs have been so great that the financial results have been far below the expectations of their projectors. Experienced engineers were not employed on many of these works; county surveyors, or in fact any one who could run a level and

discover section lines, was employed to stake out the lines, while contractors or farmers designed the dams, regulators, and other works. The grades, velocities and capacities of the canals were matters of chance or guess, rather than computation. It was not until within the last decade, when such works as the High Line and Del Norte Canals of Colorado, the Arizona Canal of Arizona, and the Calloway Canal of California were constructed, that experienced engineers were employed to design such works. In fact it would be an error to say that the men who built those works were experienced irrigation engineers, for while most of them had much experience in other branches of civil engineering, irrigation practice was then an entirely new departure to them, and only now is the science of irrigation engineering becoming properly understood.

In the earlier works little attention was paid to the hydraulics of engineering, and the dimensions of the canals were more likely to be limited by their cost than by the volume of water available for diversion from the stream. In fact several large canals have been constructed where, had their capacities been but half that given them, they would have been large enough to discharge all the water perennially flowing in the streams from which they were taken. One notable instance of such an error is the Florence Canal in Montana, which has been constructed for 15 miles with a bed-width of 20 feet, a depth of 4 feet, and a capacity of 275 second-feet; while the south fork of the Sun River, from which it is diverted, rarely discharges during the irrigation season over 30 second-feet, to all of which there were prior appropriations by the older settlers; so that the bottom of the canal has never even been wetted since its construction in 1888.

Like everything which the Americans undertake, now that they have really begun to take an interest in irrigation development, they are bringing to bear upon it their proverbial push and energy; and the advance made in the number and magnitude of the works, is being kept pace with by the skill and intelligence displayed by engineers in overcoming difficulties, and in appreciating the science of irrigation engineering. The changes wrought in the practice of this science in the past few years are astounding; there are to-day under construction numerous irrigation works, both for the utilization of the perennial flow of the streams by direct diversion and for saving the storm flow of intermittent streams by means of storage. Of these a dozen canals are completed or under construction which have capacities varying from 1 000 to 2 000 second-feet,

with bed-widths of from 50 to 70 feet, the length of the main lines of which are from 50 to 100 miles, with as many more miles of laterals and distributaries. Such canals will irrigate from 100 000 to 150 000 acres each, and will render habitable twice that area, affording at an average of 40 acres to a farm, homes and support for 3 000 families each. Of storage reservoirs, there are half a dozen completed or under construction, which will impound from 800 to 300 000 acre-feet of water each, or sufficient to irrigate and reclaim nearly that many acres of land.

While not of such magnitude as those of India, the irrigation works of the United States are second in importance to those of no other country in the world, and in many respects compare very favorably with the Indian works. The area of land commanded by works either completed or under construction, is second only to that in India. Except the Cavour Canal in the Piedmont, in Italy, there is no work of this kind in Europe which compares in size with our more modern works.

There are four essential points of difference between the irrigation works of this country and those of India. The first relates to the ownership and legislation; the second, to engineering; the third, to construction; and the last to superintendence and maintenance. In India all land and all water is the property of the Government, and the irrigation works are designed, constructed, maintained and operated by the Government. In consequence of this the legal questions involved are comparatively simple; they relate chiefly to the amounts of water to be distributed to consumers, and right of way through improved land. The question of profit is not always paramount, and while the direct money return is often small, the indirect return to the Government is always large ; in enhanced revenues from the rental of land, in immunity from famines and their consequent heavy drain on the Treasury for relief and charity, and in the general benefit to the Government resulting from increased resources and exports. In the United States, while the general Government benefits, as does that of India from the last cause, the owner of the irrigation works does not, as he is invariably a private individual or corporation. Then, as the lands are all private property, and the water the property of the public until appropriated, the constructors do not benefit by enhanced value of the land unless they purchase and own it. The priority of right to appropriate water and the ownership thereof, give rise to some of the most troublesome

and expensive legal complications with which the Western people have to deal. Direct money profit is essential to any irrigation project in this country, and in our most successful works, that has been chiefly realized from the sale and ownership of land, the value of which has been increased by furnishing it with a water supply.

As to the second point of difference, which relates to the engineering, the character of the Indian rivers and their relation to the irrigable lands is such, that the canals taken from them rarely require long or difficult diversion lines to bring the water to the lands. The chief engineering difficulties encountered are in the construction of stable diversion weirs and headworks in the sandy river-beds, and in contending against the enormous flood discharges of those rivers. In most portions of our country, good firm rock, or heavy gravel and clay soils can be found, in which to locate the headworks; while the relation of the rivers to the irrigable lands is generally such that long and difficult diversion lines have to be constructed before the water becomes available. In other words, the chief skill of the Indian engineer is in greatest demand in the location and construction of the headworks; that of the American engineer, in building the first dozen miles or so of his canal line. The only Indian canal of magnitude having a difficult diversion line is the Ganges Canal, a description of which was given by the author in an article entitled "Irrigation in India," published in Volume XXIII of the *Transactions* of this Society.

In the first 20 miles of the Ganges Canal four large flood-torrents are crossed, two of which are passed over the canal in masonry super-passages, one is admitted into it by a level inlet, while the fourth is crossed by an embankment or terreplaine and high aqueduct. Throughout its entire length, however, this canal is excavated in gently sloping or nearly level country. In the United States are numerons canals which are constructed for miles on steep hillsides and mountain slopes, often excavated in the sides of the rock cañons, requiring great lengths of rubble or masonry retaining walls to hold them up. Many deep and wide river valleys are crossed, some requiring aqueducts or flumes as great as 50 to 100 feet in height, while numerous tunnels have had to be pierced in the steeper slopes of the cañon walls. The most notable diversion line in the world is, perhaps, that of the Turlock Canal in California, which will be described at length further on. Only in extreme southern New Mexico, Arizona and California are the streams like those of India and

the dams, more difficult of construction, while the diversion lines are simpler.

The third point of difference, which is one of a structural nature, is chiefly due to the haste demanded and the cheapness required in the first cost of constructing these works. In India the works are constructed by the Government, and are accordingly designed with a view of making them so permanent that they will last nearly for all time, and that when once constructed the charges for maintenance and repairs shall be a minimum. To this end, all irrigation projects are carefully thought out and surveyed and resurveyed, until the best possible location has been chosen and the details of the scheme perfectly elaborated. The weirs, headworks, aqueducts and regulating gates are all constructed of the most substantial masonry, and the repairs to such works are reduced to a minimum. With us the main object is to reduce the first cost to the lowest possible figure, in order that the works may be completed quickly and cheaply, and the investors begin to derive some profit at the earliest possible date. The diversion weirs have generally been constructed of cheap wooden framing, and occasionally, as an unusual extravagance, of rock-filled crib-work; while many canals, especially in California, are taken directly from the streams by a simple inlet, with no dam or other diversion weir. Little time is spent in ascertaining the best location either for the headworks or the canal line, and wood is universally employed in the construction of flumes or aqueducts, regulating gates and falls. A canal may be run along the top of a fill rather than make a short detour to keep in cut, though the fill may be washed out by the first flood or be destroyed by the canal itself. A notable instance of the rapid construction of an irrigation canal is that of the Del Norte Canal in the San Luis Valley in Colorado. This canal is 60 feet wide at the bottom and 5½ feet deep. Over 30 miles of its main canal, and as many more miles of distributaries, in addition to the diversion weir and the various regulating gates and other works, were constructed in about one hundred and ten days in the winter of 1883. Many miles of this canal were surveyed and located with a construction party immediately following them. Fortunately the country is such that few mistakes were made, and the canal is a rare instance of such hasty work being done in a satisfactory manner.

The pressure which engineers are now bringing to bear upon the projectors of irrigation works is such, that more good, substantial and

permanent work is being done than at any time in the past history of irrigation development in this country, and the date is not far distant when our irrigation works will be designed and constructed with the care they deserve. There is even less excuse for faulty and unsafe work on an irrigation system than on a railway, for, should a railway bridge be badly constructed and give way, the lives of only those few persons who are upon the train which falls through it are lost, and if the line is badly located the chief loss to the company is in the deterioration of the rolling stock and added cost of haulage; but if a dam or similar work on an irrigation system gives way, not only are the lives of those whom the floods may engulf endangered, but thousands of dollars' worth of property are destroyed; and the lack of water for the irrigation of the crops may impoverish and render destitute hundreds of families, who are dependent upon the water supply to mature their crops. In addition, the bad location of the canal line and cheap construction of works, means such enormous outlays for repairs and maintenance as will consume all the returns from water rentals, and, perhaps, run the company into debt.

The fourth point of difference referred to between Indian and American irrigation works, is in the maintenance and supervision of these works. In India the fact of the completion of an irrigation system is no excuse for dispensing with the services of the engineers. A chief engineer, with his assistants, who have charge of the various divisions of a canal, are always retained in order to keep the works up, and to design and superintend improvements. In addition to the engineer corps, are overseers and patrols, the latter having comparatively short sections of the canal which they walk daily, in order to report the conditions of the works and make needed repairs, and especially to perform police duty by preventing damage being done by heedless or vicious persons, and to keep cattle from tramping over the ditches. The superintendence by these engineers and overseers renders it possible to keep the canal up to the highest state of efficiency, and the magisterial and police powers given the engineers and patrols, respectively, enable the canal officials to arrest and punish offenders against canal laws.

With us, after a canal system is completed, the services of the engineer are usually dispensed with, and the supervision and maintenance of the works falls to the lot of some superintendent, who may be a farmer, having no knowledge of engineering and as little of the needs of the work under his charge. Gophers are permitted to burrow at discretion

in the canal banks, and small leaks go unattended, as their dangers are unappreciated; while flumes and weirs and other perishable works are permitted to go unrepaired until they are past repairing and must be renewed. Rarely is any proper system of patrol in operation; sometimes ditch-riders are employed, though usually only on the more dangerous sections of the canal. On the vigilance and skill of the patrolmen and superintendent, largely depends the successful operation of a canal system, and this branch of the service is that which is most neglected and least thought of in the administration of American canals. One great canal in Kern County, Cal., that of the Kern Valley Water Company, is a startling example of the lack of proper supervision. This canal has a bottom width of 125 feet, a depth of 10 feet of water, and originally its grade was 1½ feet per mile, with falls in order to overcome the slope of the country. These falls were cheaply constructed and were soon undermined and carried away, and the canal was allowed to deepen its bed until now its grade is 4 feet per mile, and it is in places as much as 16 feet in depth below the surface of the country. Had proper supervision been kept up the falls could have been repaired before they were too badly damaged, and the exercise of engineering control would doubtless have induced the owners to replace them after they were carried away and before the canal-bed had been badly eroded.

In America, until recently, little attention was paid to hydrographic questions involved in irrigation construction. If it appeared from a cursory observation and a few inquiries that a stream discharged sufficient water for the needs of the canal company, a canal was constructed having a capacity to use more or less of that amount, and after it had been in operation a few years, it might be discovered that the average discharge of the stream was insufficient during years of minimum rainfall. In order that money may not be wasted in the construction of such works, it is essential that the discharge of the various streams should be known for a period of years. In general, no private company can afford to await the time necessary to ascertain with any degree of accuracy what this amount is. Within the past few years good work has been done to this end by public officers. The State Engineers of California and Colorado have gauged several of the more important streams in their States, and from these and the rainfall on their basins we are enabled to obtain rough data from which the run-off of their minor branches may be ascertained with some degree of accuracy. Within the past few years,

however, the Irrigation Branch of the United States Geological Survey has inaugurated some valuable and extensive researches in this direction, and had the Government appropriations permitted their continuance for a few years longer, these would have furnished us with results of inestimable value. As it is, these observations are still being conducted, though with less detail and thoroughness than would be the case were liberal appropriations available. Regular stream gaugings were conducted during the years 1889 and 1890, on most of the more important streams in the arid regions, and, as the former of these two years happened fortunately to be one of the driest in the history of our country, we are able to determine approximately what is the minimum run-off from the catchment areas of most of those streams; while comparison with observations conducted during a period of years on certain streams in Colorado and California, shows that the year 1890 was one of good average precipitation. In addition to this good work, observations were conducted to determine evaporation, seepage in different soils, and the amount of sediment carried in suspension during floods. From these it is now possible to approximately determine the loss from evaporation in storage reservoirs in the localities in which these observations were made. In this direction, however, there remains much yet to be done. Fortunately the directors of several agricultural experimental stations, which are operated by the United States Government in connection with the State Agricultural Colleges in the West, appreciate this, and experiments are now being conducted at Fort Collins, Colorado; Tucson, Arizona, and a few other stations, from which we may hope in the course of time to learn for those particular localities what are the annual and average amounts of evaporation, both from water surfaces and from soil; what is the duty of water on different soils and crops, and the percentage of loss in canals by absorption, in addition to other similar problems that await solution. To be sure, much work of this character has been conducted by eastern engineers in connection with the water works for cities; this is notably the case with the experiments on the Cochituate and Croton watersheds, but the results obtained are of comparatively little value when applied to the changed climatic conditions prevailing in the west.

The scope of this article is such that it is not possible to give in detail the results of experiments to ascertain duty, absorption, and the effects of alkalinity, as these differ greatly with different crops, soils

and climatic conditions. In an able article on "Irrigation," read before this Society in March, 1887, Mr. Edward Bates Dorsey gave much valuable information on these subjects, while the discussion provoked by the article produced as much more. It is well to note, however, that as the influence of capitalists and educated engineers is brought to bear on the cultivators, and as the experience of the latter is increased, the duty of water is constantly rising. This result is also largely due to the increased value of water and the saturation of the soil by successive years of irrigation. In Colorado, for example, State Engineer Maxwell and others, who a few years ago placed the duty at from 60 to 80 acres per second-foot, now claim that it is 100 acres, and a few believe it to be 125 acres per second-foot. In California, Wyoming and elsewhere, the same change of opinion is taking place; and while the ordinary duty of water in the San Joaquin Valley is estimated to be 110 acres, a duty of over 1 000 acres has been obtained in a few cases in Southern California by means of sub-irrigation from pipes.

A few years ago a great outcry was raised regarding the efflorescence of alkaline salts and the production of fevers due to irrigation. This, while true, is coming to be less feared as its causes are better understood. With an increased knowledge of the necessity of natural or artificial drainage as an adjunct to any well-planned irrigation project, and a better understanding of the proper mode of locating and constructing canals, so as to produce a minimum rise of the sub-surface water level, the damage caused by these evils is reduced to a minimum.

By far the most important and interesting problem in connection with irrigation which yet awaits solution, not only in our own country but wherever in the world irrigation is practiced, is that of securing a uniform and correct standard or unit of measurement of water. At present water is not sold, as it should be, like other commodities which have an intrinsic value per pound, yard or gallon, though it is appreciated by every engineer and canal owner throughout the world that such would be the most satisfactory method of disposing of it; but owing to the difficulties of measuring flowing water at a cost commensurate with its value, some other device has always been employed, such as charging a higher rental, or a higher cash selling price, for lands to be irrigated; or putting a charge of so many dollars per acre on the land served, as a water-right, or by some similar and equally unsatisfactory method, whereby the amount of water going with the land is not specified and

remains always a point of contention between the purchaser and vendor. On this subject Mr. Walter H. Graves, C. E., one of the prominent irrigation engineers of Colorado, says:

"The difficulties attending the measurement and allotment of water by reason of the constantly varying conditions and fluctuating head, has given rise to a great variety of methods and customs; and in this respect where system is all important and most needed, the absence of it is conspicuous, and all efforts for the establishment of a correct and universal system have thus far been defeated. The most common practice in Colorado is by a sort of nondescript 'inch' unit, taken from a method of measuring common among miners in placer diggings called 'the miners' inch,' this in turn is taken from a module or unit of measure called the *oncia magistrale,* in vogue in the irrigating provinces of Italy, that was devised some three or four centuries ago by one Soldati. It is rather a method of subdivision than one of measurement. It is called the 'statute inch' from an attempt to prescribe it by State statute; the very provisions of which, however, make the incongruity manifest. In practice it is impossible for the consumer to know how much water he is using. There is a great need throughout the entire irrigating region of a simple, practical, trustworthy apparatus for the measurement of water. The water meter employed in the water-works of cities is too complicated to apply to the open ditches, where it may be choked with mud and weeds. The ingenuity of some engineer will be amply repaid for the invention of a hydrometric sluice that will meet the requirements of this demand."

A new water meter for irrigation, devised by Mr. A. D. Foote, and described by him in a paper read before this Society in March, 1887, is perhaps the best form yet suggested, and a modification of this employed by Mr. Graves on the canals of the San Luis Valley, in Colorado, seems to work with some degree of satisfaction. Next to this the common V measuring weir is one of the best devices employed.

Perennial Canals.

Of the older type of canals constructed in our country it is necessary to say little. Both in Colorado and California, irrigation by Americans as distinguished from that practiced by the original Mexican inhabitants for several centuries past, came into vogue in the latter part of 1860; when some of the early pioneers abandoned gold digging for the more profitable industry of agriculture, and when in California, especially, they discovered after repeated failure that the rainfall was deficient for crop raising. The practice thus forced upon them was at first compara-

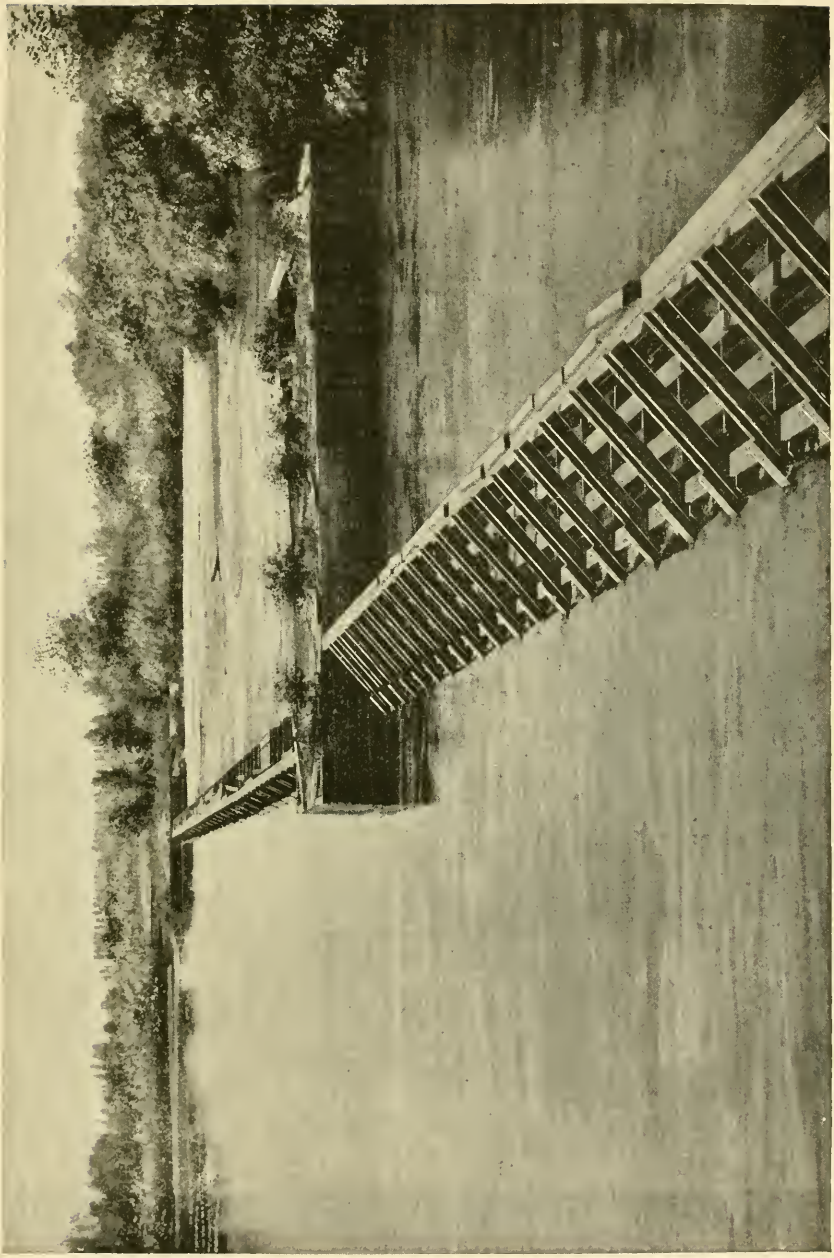
tively easy of accomplishment, for in many localities the mining ditches which were ceasing to return profit from disuse, were ready at hand for the transportation of water to small garden plots and fields in the foothills of the gold-mining region, and at once brought home a blessing in disguise. In many parts of California and Colorado the early pioneers in irrigation are known now on account of their great wealth, most of which was realized through the aid of irrigation. For several years the practice of irrigation was on a moderate scale. At first, the water was diverted from the smaller streams, which rendered it easily accessible by the individual efforts of the poorer farmers, and each irrigator owned his own separate ditch, was his own engineer, and himself constructed the works. The dams and head-works usually consisted of brush or stone thrown across some narrow channel; and the canal, of a straight cut, without any regulating gate to control the water, badly aligned, having accidental slopes and no pretence at uniformity of cross-section or grade. As the water was easily obtainable, however, such works proved profitable and beneficial. As the water of these smaller streams became all appropriated, and later settlers were compelled to look to larger streams for their water supply, co-operation for the construction of larger weirs by regular ditch companies became a necessity. Of these earlier works of greater magnitude, among the first constructed were the Fresno, King's River and San Joaquin Canal, and a few others built in the San Joaquin Valley.

The first of these was constructed in 1872 by Mr. Isaac Friedlander of Fresno County, under the direction of Mr. Alfred Poett, Civil Engineer, and was perhaps the pioneer of this kind of work in our country. The Fresno Canal is diverted from the Fresno River by means of a timber dam, 311 feet long, which raises the water 6 feet above its original level. It was constructed of two rows of main piles, from 20 to 35 feet long, planted 10 feet apart and driven firmly into a stratum of clay. Between the piles was a double row of 4-inch sheet piling, and the space between the two rows of piles was filled in solid and planked on top. Below the toe of the dam was a timber apron of 4-inch plank set at an incline, from which the water passed on to a thick layer of loose rock. The head-works of the canal consisted of a sort of flume or box of timber 30 feet wide and closed by six gates. The canal had a uniform bed-width of 20 feet with side slopes of 1 on 2, the depth of the canal being 8 feet and the depth of water 6 feet. Where they were

in embankments the top width of the canal banks was 4½ feet. The grade of the canal as originally designed was 6 inches to the mile, and it was designed to give a velocity of 2 feet per second and a discharge capacity of 360 second-feet.

The heads of the distributaries consisted of an arrangement of fluming or boxing closed by simple sliding gates. The main distributing ditches were 10 feet wide at the bottom, slopes 1 on 2, top width of banks 2 feet, and height 2 feet, with varying grades, the maximum grade being 33 inches to the mile and the velocity about 2¼ feet per second. In designing the above work it was estimated that this canal would irrigate 72 000 acres, and the duty of the water was expected to be from 200 to 400 acres per second-foot. It is an interesting and notable fact that, whereas the duty of these earlier canals in California did not rise much above 80 to 100 acres, experience in its use and its increased value are daily causing this duty to increase, so that now it not only reaches the earlier estimates but in many cases even passes them. Among the weak points in earlier irrigation construction were the steep grades and high velocities given, and practice is now bringing these down to the theoretical dimensions designed for the Fresno Canal. The earlier canal works of California were faulty in many respects, but chiefly in the location and construction of their head-works. In these, if any weir at all were built, this was done in an unsubstantial and cheap manner, causing great loss both in the operation of the canal and in the necessarily frequent reconstruction of the weir. In all cases the alignment of these canals was faulty; sometimes sharp bends were made, which greatly retarded the flow of the water and caused the deposit of sediment and erosion of the canal banks, while other constructors went to the opposite extreme, giving great broad curves, such as would be necessary on a railway line. No provision for the discharge of flood drainage was made, and the canals were frequently destroyed, owing to their gathering local flood waters for which no relief was provided; while grades were equally faulty, the chief error being in making them too steep.

The next stage in the development of canal evolution is represented by such works as the Del Norte and Highline Canals in Colorado, the Calloway Canal, in Kern County, Cal., and the Arizona Canal, in Arizona. Though these works were in general fairly well aligned, and while intelligent thought was brought to bear on their design, they show, however,



KERN RIVER DIVERSION WEIR, HEAD OF CALLOWAY CANAL, CAL.

a lack of experience on the part of the engineers constructing them and of the capitalists furnishing the money, which was due to the newness of this class of work. Their chief fault lies in the perishable character of the work done upon them, especially in their head and regulating works, which are of the most temporary character.

The Calloway Canal is diverted from the north bank of the Kern River, a short distance above the town of Bakersfield. It was located in 1875, and appropriates 1 476 second-feet of water, though its capacity is little over half of this. The Kern River is the third largest in the San Joaquin Valley and rises in the high Sierras. Its headwaters drain the base of Mount Whitney, whose melting snows insure it a perpetual discharge of some magnitude. The average discharge of this river is about 2 700 second-feet, while the average maximum discharge during the rainy season is 19 000 second-feet. The water of the canal is diverted from the river by means of an open wooden weir 400 feet in length and extending across the entire width of the river from bank to bank at right-angles to its course. This weir (see Plate XXVI) rests on three rows of 4 x 12-inch anchor piles, driven 10 feet into the river bed, and two rows of 4 x 12-inch sheet piling parallel to the course of the stream. There are one hundred rows of these piles across the bed of the stream and upon them rests the superstructure, consisting of as many bays, 4 feet in width between centers, and composed of beams or rafters set at an angle of about 40 degrees facing up-stream, and 50 degrees down-stream. On the bed of the river between these bays and resting on mud-sills laid on the tops of the piles, is a 2-inch flooring about 30 feet in length laid with the direction of the river. On the upper sloping face of these open timber bays are set grooves into which slide 2-inch planks, or as they are commonly termed "flashboards," and these are inserted in sufficient numbers until at low water they fill the weir from base to crest, forming a continuous closed dam. At high water and in flood times these planks are removed, one at a time, thus increasing the water way of the flood to any desired amount. The total height of the weir is 10 feet above its floor, and adjacent to its north end and closing the head of the canal, which is diverted at this point, is an open wooden regulator constructed in almost exactly the same manner as the weir, but exceeding its height by 1 foot.

For the majority of its length the Calloway Canal is excavated in a light, sandy loam, which is exceedingly rich and fertile, but of so light a

character for construction that the slopes given the banks are necessarily low. The canal line skirts around the slopes of the foot-hills to the northward on a changing grade, the object of which is to diminish its discharge as the various distributaries are diverted from it, while leaving it the same cross-section in order that it may ultimately be enlarged. The change of grade is from 0.8 feet per mile to the tenth mile, to 0.6 feet, then 0.4, and then it is level near the end. The capacity of the canal at its head is 700 second-feet, its bed-width is 80 feet, depth 5 feet, depth of water 3½ feet, and maximum velocity 2½ feet per second. Throughout its length the canal is built half in cut and half in fill. Its side slopes are 1 on 3 inside and 1 on 2 outside, though owing to the light, sandy character of the soil, these have changed until now the interior cross-section is an easy curved slope from the top of the bank to the center sub-grade. The canal is 32 miles long, and at its second mile a double escape and regulating head is constructed for the purpose of discharging all the waters at this point in case of necessity. This consists of weirs which, like all the others on its line and that of its distributaries, are constructed of wood after the general design described for the head weir and regulator. In the thirtieth mile, Poso Creek, the first principal drainage encountered, is admitted to the canal by a level inlet, while in the opposite bank of the canal a wooden outlet weir is constructed of the design above described. Diverted from the Calloway Canal are some sixty-five distributing ditches, from 8 to 20 feet bottom width and from 1 to 9 miles in length, their aggregate length being 150 miles. These branches will carry 3 feet in depth of water, and have slopes the same as that of the main canal and a grade generally of 1.6 feet per mile. This canal commands 200 000 acres of land and most of the water is now employed for the irrigation of the numerous ranches below it. On the line of its distributaries, which run down the slope of the country, numerous falls are constructed, their design being similar to that of the head weir, and their object being to maintain a uniform grade throughout the canal and keep its waters as near the surface of the irrigable land as convenient.

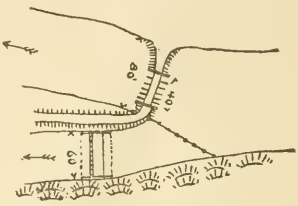
There are some peculiar features connected with the construction of the Del Norte Canal, which is situated in the San Luis valley in Southern Colorado. This canal takes its supply from the Rio Grande at a point just north and west of the town of Del Norte, and after skirting the

foot-hills north of the river for a few miles, it is located around the northwestern edge of the valley, and commands most of the lands between the Rio Grande and Saguache Creek, of which it will irrigate about 225 000 acres, though the area commanded is much larger than this. The discharge of the Rio Grande, as shown by gaugings maintained by the hydrographers of the United States Geological Survey at Del Norte, averages about 1 250 second-feet; and while its minimum discharge has been as low as 400 second-feet in the month of August, its maximum has reached at least 5 000 second-feet.

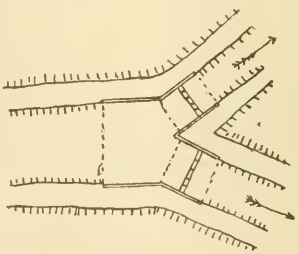
One of the points of interest in the construction of this canal was, as before stated, the rapidity with which this work was done. In the four winter months, from December to April, 1883–84, something over 1 400 000 cubic yards of material were excavated to form the channel, requiring the employment of nearly 5 000 laborers and 1 200 teams, all in a sparsely inhabited country. The location of the headworks was not the best; they should have been placed at least 10 miles further down the river, according to the statement of Mr. Walter H. Graves, the engineer who built them. As the canal was projected by the citizens of Del Norte, local pride prevailed over good sense, and decided upon their situation adjacent to the town. The excessive grade consequently given in the first few miles of this canal is another curious and noticeable feature of its construction. To have adopted a grade dictated by good judgment would have rendered its construction impracticable, as it would have climbed to the tops of the adjoining mountains before reaching the lands to be irrigated, so great is the fall of the country to the north and east, hence the line skirts the base of the foot-hills for the first 12 miles, and the canal is given a fall corresponding with the natural slope. This grade is excessive for so large a canal, averaging about 8 feet per mile until it emerges from the foot-hills, where it is reduced to about 1 in 2 112. It was anticipated that though the material through which this canal was constructed was entirely of coarse gravel, drift and rock, it might be necessary in time to revet its channel with rock, as no falls were put in it. Fortunately this has proven to be unnecessary. In one place the grade is 35 feet per mile for a quarter of a mile, but as the canal here is straight little damage has been done by this high velocity.

The diverting weir, of which a sketch plan is shown, is located at a point where the Rio Grande flows through a comparatively narrow

gorge having firm earth banks on one side and a rock bluff on the other. This weir has a total height above the grade of the river bed of 5 feet 6 inches. It consists of a series of open bays which are closed by flash-boards, and is almost identical in construction with the weir at the head of the Calloway Canal, though a little more substantial than that weir. Its total length is 80 feet, and it contains seven openings of 6 feet wide each, the remainder of the weir consisting of a rock and earth embankment, over which it is not intended that the flood waters shall pass. As there are several separate channels to the Del Norte River just above the canal-head, it has been necessary to construct wing dams and other obstructions for the purpose of training the main body of the river into the single channel which passes by the head of the canal. This is done by means of wing dams of earth or rock, and in one case by the construction of a large crib dam entirely closing one channel. This dam is constructed of 12 x 12-inch crib work, drift-bolted and wired together and filled with rock. Its height varies from 6 to 15 feet, while its length is 1 100 feet.

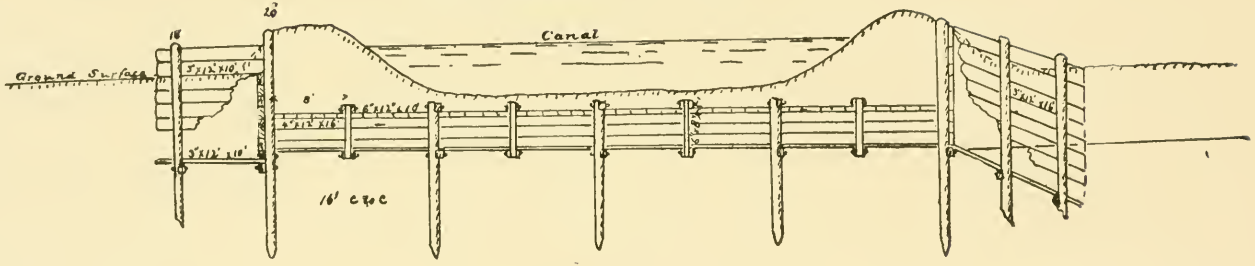


The regulator (see Plate XXVII, Fig. 2) is situated back in the canal a hundred feet or so below the entrance cut, and consists of a wooden structure having a heavy plank flooring resting on piles driven in the canal bed, and having wooden wings along its sides for protection from erosion. It consists of ten gates, giving a width of 59½ feet of clear opening, and 8 feet in height. These gates slide vertically between guide posts, and are operated from above by means of screws turned with hand levers. For the first section of the canal below the head the bed-width is 60 feet, depth of water 5½ feet to grade, below which is a sub-grade of 1 foot. Its maximum capacity is 2 100 second-feet, which is the second greatest of any canal in this country. Eight thousand feet below the head is a bifurcation dividing the canal into two equal branches, each having a bed width of 40 feet. This bifurcation, of which a diagram is shown, consists of wooden planking protecting the outer sides and the dividing point

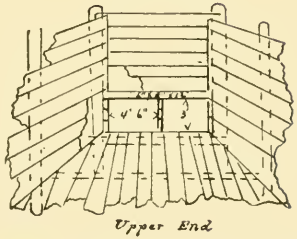


Box Culvert, Del Norte Canal

LONGITUDINAL SECTION



END VIEW



CROSS SECTION AND ELEVATION

Regulating Gates Del Norte Canal

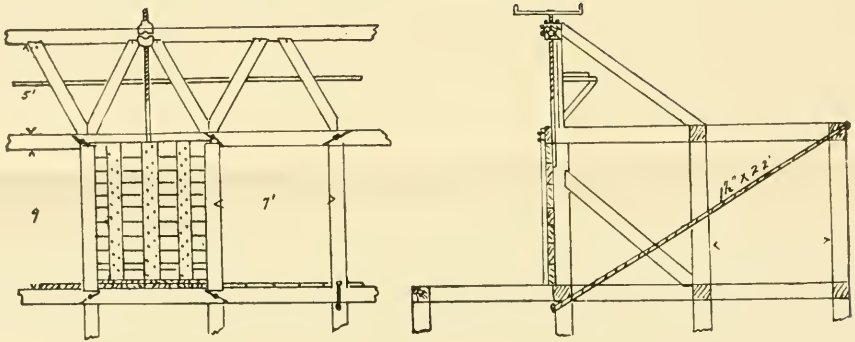


PLATE XXVII.
TRANS. AM. SOC. C. E.
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WILSON ON AMERICAN IRRIGATION.

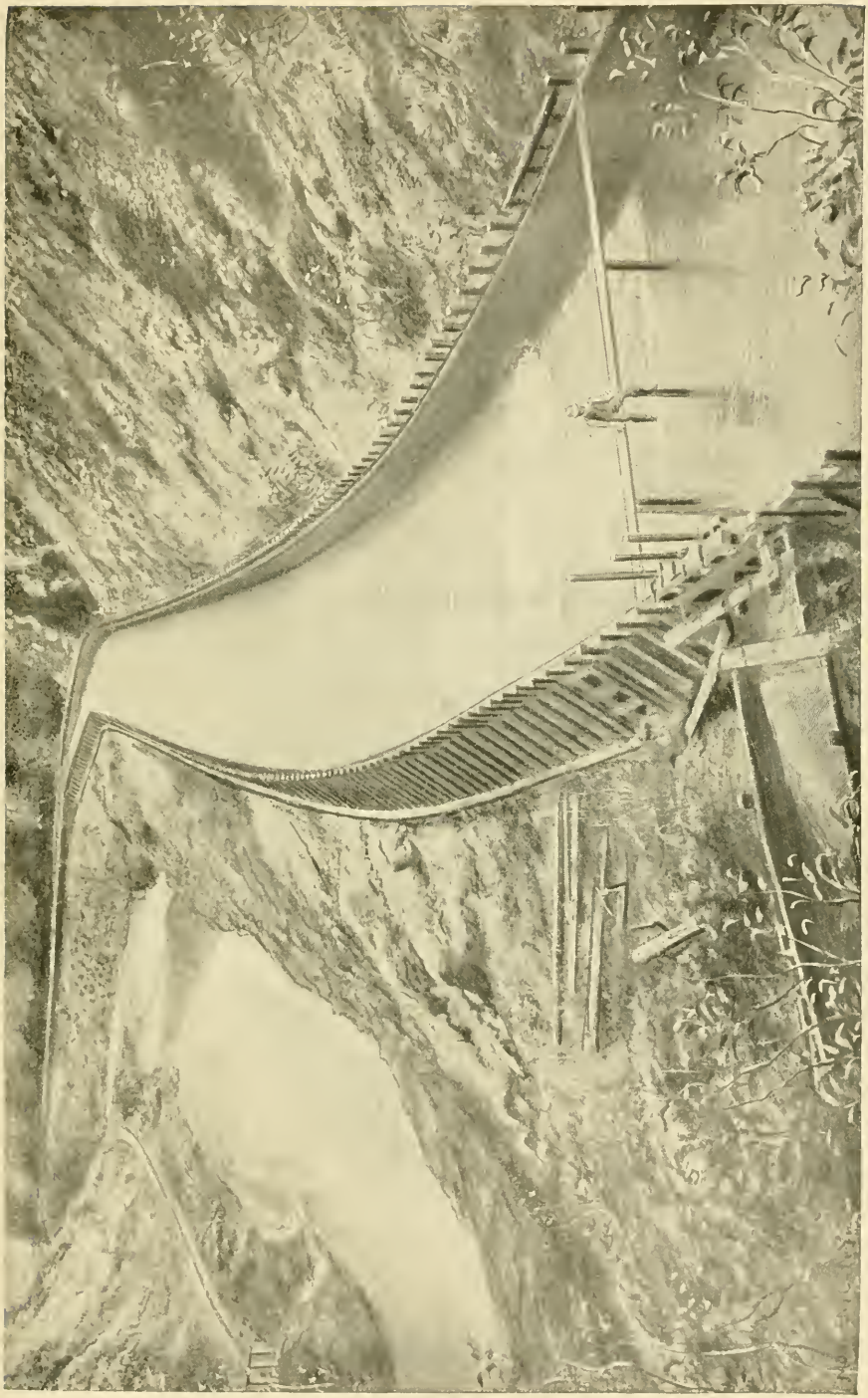
between the two canals, and of a wooden flooring resting on piles. In the head of each branch is a regulator by which the discharge into either can be controlled, and each regulator consists of five gates of the same general dimensions and form of construction as that at the head of the main canal. Lower down in the south branch is another bifurcation similar in character to that above, while numerous distributaries and smaller laterals are taken from both branches in such manner as to command most of the land below the canal and supply the private ditches. The fall in the majority of the branches of this canal is great, being from 10 to 15 feet per mile. When in gravel this has caused little damage, but the majority of these distributaries are in earth excavation or embankment, and have sustained much damage by erosion, necessitating the rip-rapping of large portions of their length. The engineer in charge of the work says, however, that it is cheaper to do this than to give the increased cross-section necessary with a less grade and velocity.

On the line of this canal are several works interesting in their design, but simple in construction. One is a wooden aqueduct whereby the Farmer's Union Canal, a separate irrigation channel, is carried over the south lateral of the Del Norte. The grade of this canal is kept high enough to give a clear crossing of the Del Norte Canal, immediately after which the water is dropped to the general level of the surrounding country, in two falls of 3 feet each, constructed of wood. In another place a smaller canal is taken under a branch of the Del Norte by means of a wooden box culvert or inverted siphon. (See Plate XXVII, Fig. 1). This culvert is founded on piles, and contains two openings of 3 x 4½ feet each lined with 4-inch planking, and covered on top by an additional depth of 6-inch timbers to support the weight of the superincumbent earth and water. The cross-section of this canal is unlike that usually employed on canals in other countries, and in most parts of this country, owing largely to the light and sandy soil in which it is excavated. The banks have no set top width nor fixed outer slope, though this latter is about 1 on 3; the inner surface has side slopes of from 1 on 2½ to 3, and is made deepest in the center, while the bed is not level, but is sunk to a sub-grade of about 1 foot in depth.

The Highline Canal of Colorado is another of the earlier canal structures of this country. It is diverted from the South Fork of the Platte

River, about 21 miles above Denver. Whereas the maximum flood discharge of this river is about 2 000 second-feet, its minimum flow averages as low as 110 second-feet; and, as a consequence, though the cross-section and discharge capacity of this canal is large, the water supply is not equal to the demand during a majority of the irrigation season. This fault could, however, be easily remedied by the construction of storage reservoirs on the Upper Forks of the Platte River. The canal commands an area of about 90 000 acres of excellent irrigable land in the neighborhood of a populous city, and should be one of the best canal properties in the country if its water supply were but secured against drought. This canal was designed and constructed by Mr. Edward S. Nettleton, though Mr. George G. Anderson, who has been the engineer in charge of it for the past several years, has made some marked changes and improvements in it, especially in reconstructing more permanent head-works.

The diversion weir is 14 feet in maximum height, 117 feet long and 6 feet wide on top. It consists of crib-work filled with rock, and whereas the up-stream slope is very flat, down-stream the fall is nearly vertical. Near the left bank is constructed a substantial masonry undersluice, open the entire depth of the weir, whereby the water in the river above may be drawn down to any desired level. The sluiceway consists of four iron gates, each 4 feet wide between centers and 7 feet in height. These gates are raised by means of screws, and the sills of the gates are placed at a distance of 2 feet below the level of the head-gates of the canal, in order to permit of sluicing out sand, etc. The canal is diverted from the right bank just above the weir, the head-regulator consisting of a set of five gates, each 6 feet in height and $3\frac{1}{2}$ feet wide between centers. This head-regulator is constructed between solid rock walls and on a rock bed, and the gates, which are of wood, are raised by screws from a platform above. Immediately below the regulating gates is a tunnel excavated in granite, 600 feet long, 20 feet wide and 10 feet in height in the center; the lower end discharging into a great wooden bench-flume, which skirts the steep rocky slope of the Platte Cañon for a distance of 2 640 feet, where it terminates in the open canal excavation in which the water is carried throughout the remainder of its course. At the head of this flume (see Plate XXVIII) and at the lower end of the tunnel is an escape by which the water carried in the canal can be regulated. This escape consists of five gates, each 3 x 4 feet, of wood, and raised



BENCH FLUME ON HIGHLINE CANAL, COLORADO.

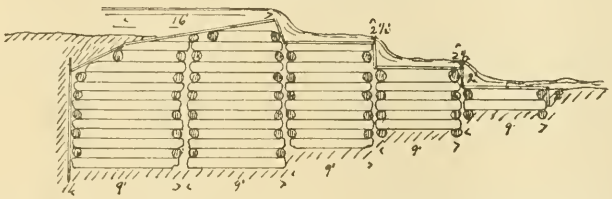
by rack and pinion, while immediately below these gates, across the entire width of the flume, are a set of check or flash-boards about 2 feet in height, which act as a sand-gate, by causing the deposit of silt immediately above them, whence it can be scoured out through the escape gates.

The great bench flume which extends for over half a mile below the tunnel, is 28 feet wide and 7 feet deep; its grade is 5.28 feet per mile, and its discharge, which is the same as that of the canal proper, is 1 184 second-feet. The main canal is about 85 miles long, and bifurcates at the 51st mile, one branch being 14 miles long, and the other 20 miles long. At its head the main canal is 40 feet wide on the bottom, and from 7 to 8½ feet in depth, with slopes of 1 to 1 in cut, and 1 on 2 in embankment, and with a uniform grade of 1.76 feet per mile. The line of this canal is crossed by several side gulches or torrents. In the first mile is an embankment 18 feet in height at a gulch crossing, but as the discharge of this is small, there is no inlet nor outlet dam constructed for the regulation of its flow, the water being admitted into the canal. In the third mile, Willow Creek is crossed by the construction on the lower side of the canal of a heavy earth embankment, which is 30 feet high and 8 feet wide on top, with slopes of 3 to 1 inside, and 2 to 1 outside. Above and below this embankment, escapes are constructed in the canal banks, which discharge directly over the natural gravel slopes of the country into Willow Creek. These escapes have lengths of 200 and 300 feet respectively. At the 9th mile, Plum Creek is crossed by a flume 918 feet long and 18 feet in height on a high trestle, founded on eight rows of 12-foot piles. In this flume are four sets of escape gates spilling into Plum Creek, and by which the discharge of the canal can be regulated. Originally a less substantial weir existed at the head of this canal, which was washed out and replaced in 1884 by the present weir. The total cost of both weirs, flume, ditch and other works was about $650 000, and the maintenance, cost, charges, etc., amount to about $25 000 per annum, including salaries, office expenses, etc.

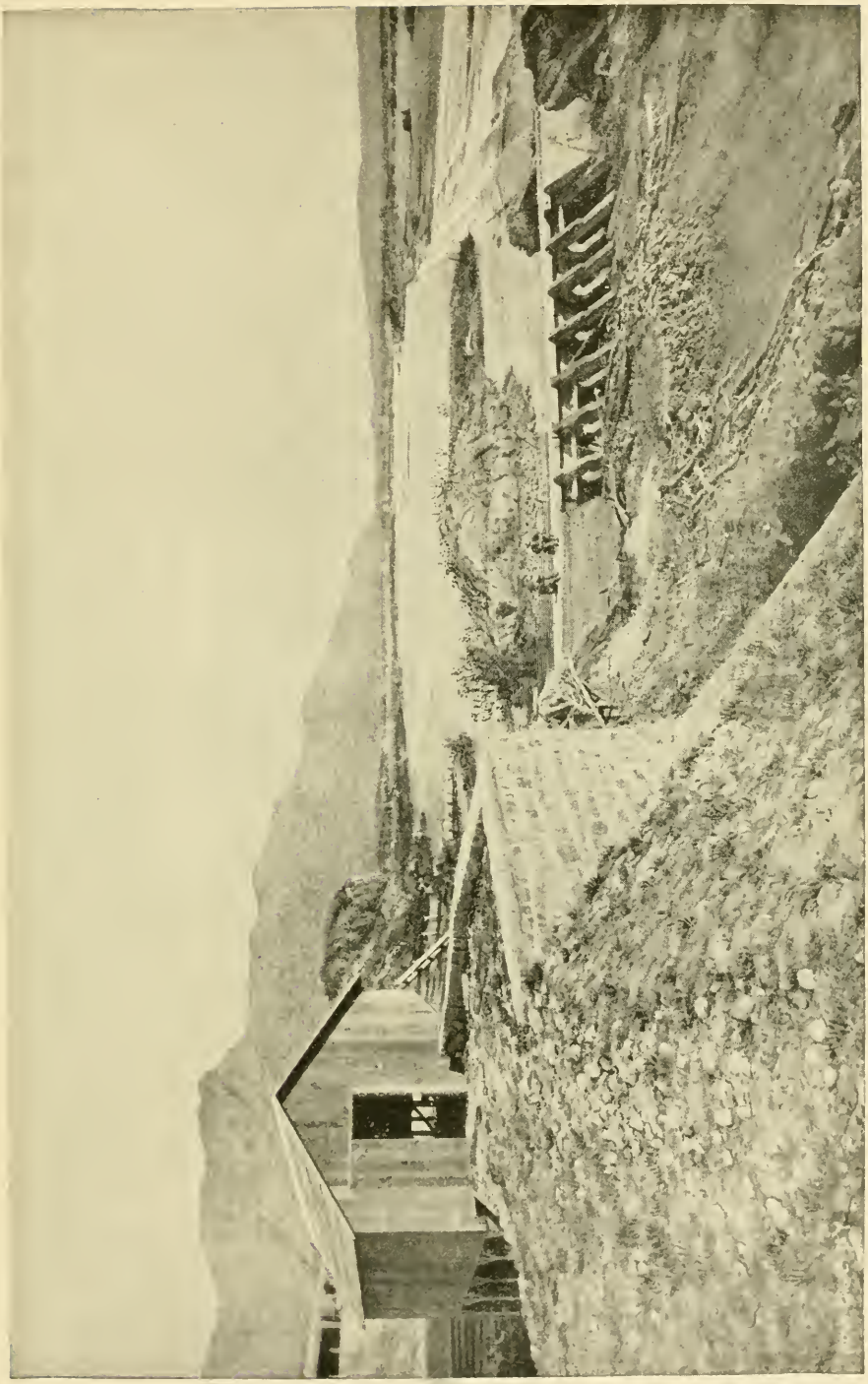
Another equally fine example of the third stage of canal construction in this country, in which stage are included all those large canals which have been put in operation during the past few years, and similar in most of its features to the works just described, is the Arizona Canal, which is diverted from the Salt River in Southern Arizona, about 25

miles above the town of Phœnix. The head of this canal is situated at a point 1 mile below the junction of the Salt and Verde Rivers, where the river leaves the Lower Cañon, and flows between a reef of rocks which affords an excellent site for the weir and headworks. Thence its course is along the north bank of the river, skirting the foothills surrounding Salt River Valley, and commanding an immense area of the finest quality of rich arable land; where, owing to the semi-tropic character of the climate, abundant yields of hay, grain, vegetables and such fruits as plums, peaches, grapes and oranges are cultivated. The Salt River is an extremely difficult stream from which to divert a canal, owing to the irregularity of its discharge, for while its average minimum flow is usually as low as 500 second-feet in midsummer, the greatest recorded flood was 300 000 second-feet, and each and every year floods from 10 000 to 20 000 second-feet occur. As a consequence of this erratic discharge, the river bed itself is very wide, and a long and expensive diversion weir is required in order to procure stability and permanency.

CROSS SECTION
ARIZONA WEIR



The weir at the head of the canal (see cross-section and Plate XXIX) is constructed of crib boxes of rough hewn logs about 9 feet long each, drift-bolted, wired together, and filled with rocks. The length of the weir is 916 feet, and it is set at an angle with the current of the stream, the object of which was to procure the most direct line of location between rock abutments, while the result is to force the water over toward the canal-head in a manner which is not altogether safe and satisfactory. The total height of the weir from the bed of the river to its crest is 10 feet for a little over half its length from the end adjacent to the canal-head, and 11 feet for the remainder of the way. The total length of the weir and apron parallel to the course of the stream is 48 feet, and in the deepest part of the gravel bed of the river the depth of crib-work is 33 feet. The up-stream side of the dam is planked, as is its entire facing,



WEIR ACROSS SALT RIVER, AND HEAD-WORK OF ARIZONA CANAL.

and is on a flat slope inclining backward from the crest; while sheet piling is driven in above the crib-work on the upper face. Down-stream the crest of each successive row of cribbing, as shown in the accompanying diagram, is from 2 to 2½ feet lower than that above it, causing the weir to fall away in broad steps, each of which is about 9 feet in length horizontally, thus breaking the force of the falling water and diminishing its tendency to destroy the weir; the last and lower line of crib-work being practically a swinging apron resting on the river bed and wired to the crib-work above it. This weir has proven very substantial, and has withstood for four years the enormous flood volumes above indicated. In the spring of 1891, however, the big flood of 300 000 second feet did much damage to it, and a new weir is now being constructed at the upper site, described further on. Beyond the east end of the weir (that furthest from the head of the canal), is an old channel of the river, separated from the present channel by a narrow ridge of rock and gravel, a few yards in width. This old channel is blocked by an embankment of gravel 10 feet in height, rising to an elevation a little above that of the crest of the main weir. The location of the head-works is not the best that could have been discovered, but their choice was forced upon the engineer, Mr. Sam A. Davison, owing to the limited time at his disposal in which to construct them and the smallness of the funds. By the selection of the present site he was enabled to build the works in much less time and at less expense than by the choice of a better site. It is proposed, however, to at once re-locate the dam at a point a couple of hundred yards higher up the stream, where a shorter weir can be constructed between better abutments and on a rock foundation. This will enable the weir to cross the river at right angles and permit the head of the canal to be diverted at such an angle from

the weir as will give a clear scour past the regulating gates, thus reducing the pressure on the head and avoiding the objectionable deposits of silt in front of the gates which now occur. (See diagram.)

At present the head of the canal is diverted at a point about 50 yards above the end of the weir from the right or western bank of the river, and is separated from the end of the weir by two rocks which project above the water surface, and between which a waste or scouring gate has been constructed, the object of which is to prevent the deposit of silt in front of the canal-head. The relative position of these works is such, however, that quite an island has formed almost in front of the head-gates, which interferes with the free flow of water into the canal. The scouring sluice between the weir and the regulator is a simple structure constructed of wood, founded on piling and closed by flash-boards, and is very similar in its general design to those described for the open portion of the weir at the head of the Del Norte Canal. The regulating gates at the head of the Arizona Canal are well and substantially constructed of wood, and over the top of them crossing the canal from bank to bank, is a covered bridge giving ample room in which to operate the gates. These latter are eight in number, and are each 4½ feet wide by 6 feet high, though the total height of the regulator head is about 11 feet in order to prevent its being topped by extreme floods which bear with considerable pressure against the gates. Of the regulators six are raised by simple hand-levers, which are inserted into holes in the upright rods attached to the gates, and are used when the river and the pressure are low. Two of the gates, however, are operated by screws ingeniously geared by cog-wheels in such manner as to be operated by a winch turned by hand, and can be worked by one man under the greatest head of pressure.

At its head, and for the first considerable portion of its course, the canal has a bed width of 36 feet, carries a depth of 7½ feet of water and has a capacity of 1 000 second-feet. It is laid out with a uniform grade of 2 feet to the mile, its banks having slopes of 1 on 1½ in rock, and its cross-section being similar to that of the Del Norte Canal, with no berme, and a sub-grade of about 1 foot in depth. The main line of canal is 41 miles in length and it is now being extended. For the first 3½ miles below its head, excavation is entirely in rock or gravel, in places the gravel cut being 25 feet in depth. Below this heavy work the canal is entirely in earth on very gently sloping side hill, and is built half in excavation

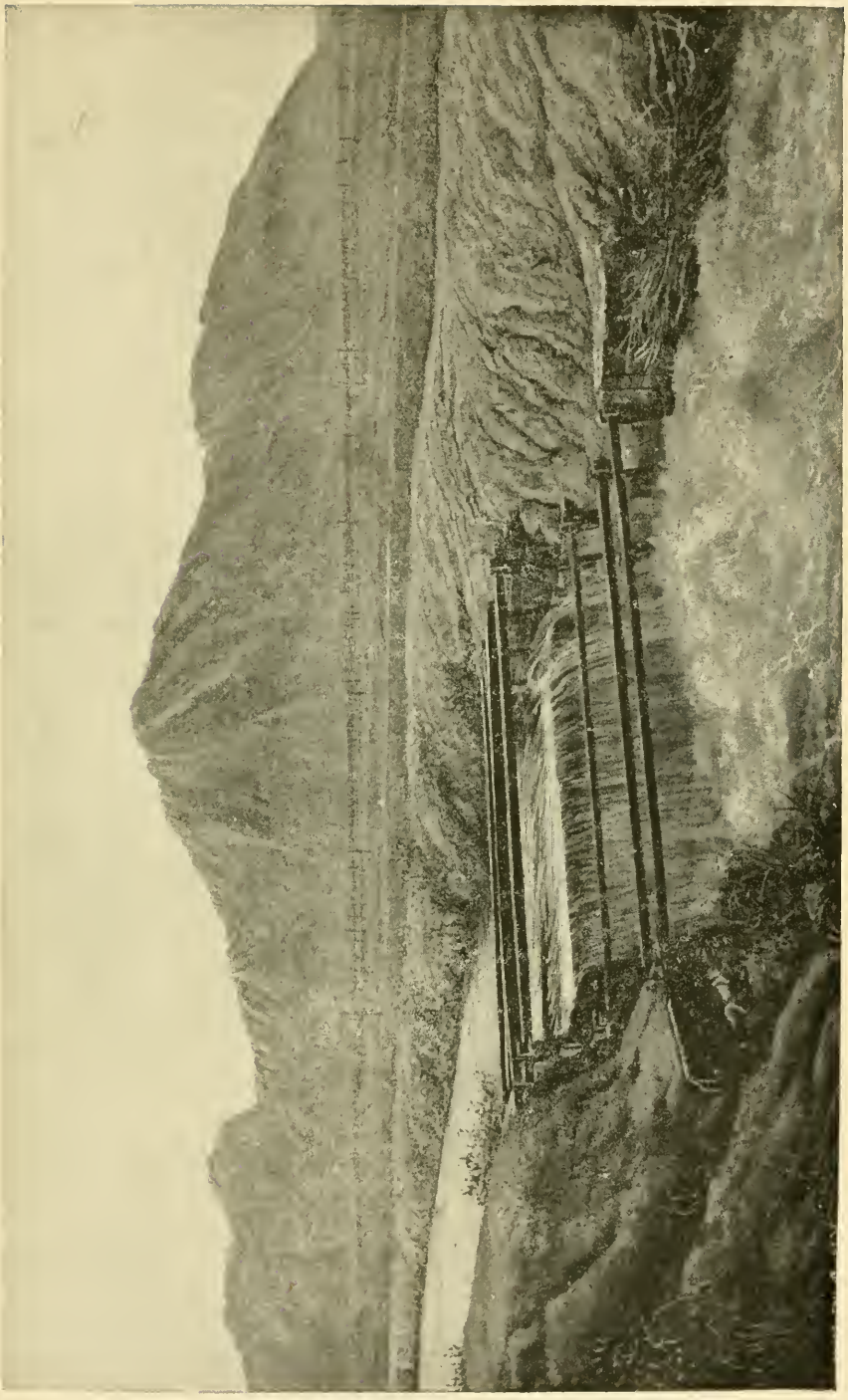
and half in embankment, with the exception of one short rock cut just above the diversion of the cross-cut canal. This cut is 15 feet in depth and has a fall built in solid rock 15 feet high, constructed both in order to drop the grade and to avoid some rock excavation, and also with a view to being used for furnishing water-power to be employed in the town of Phœnix. About 1 mile below the head of the canal an escape has been constructed, by which the discharge water can be let back into the river through a waste channel having a bed width of 40 feet and a length of 800 feet. The escape regulators consist of a row of gates having a total width of 40 feet, a length of 80 feet, and a depth of 12 feet, with a strong apron at the upper end extending at an angle of 45 degrees downward for 12 feet into the bed of the canal. Across the canal itself, below and adjacent to the escape gates, is a set of regulating gates constructed in a similar manner to the escape gates; the whole work rests upon piles, is well floored, and has stout wings to protect the banks. Well planned provision is made for sluicing out and getting rid of any sand which may deposit in the channel of the canal, and of emptying the canal in case of accident below.

After it reaches the earth excavation below the heavy work, the canal has a uniform grade of 1 foot to the mile and is 30 feet wide on the bottom for a distance of 26 miles, having an 8-foot berme on the embankment side, the banks having slopes of 1 to 1 in cut, and 1½ to 1 in embankment. In all cases the top width of the bank is 8 feet, and its crest is from 6 to 8 feet above the bed of the channel. All fills have extra widths and heights, giving them a secure appearance, while the curves are generally made very favorable and have been laid out with some attempt at intelligent alignment. In the first few miles the canal crosses several small drainage streams which have flood discharges of from 30 to 200 second-feet. These streams are admitted by simple level inlets, the water being passed on down the canal without any provision for wastage or escape, which is an undoubted defect in the designing of the work. Beyond Phœnix is a broad, level stream bottom of considerable extent, through which Cave Creek finds its way, having a flood discharge of at least 1 000 second-feet. At present owing to the difficulty and expense of confining this stream to a single channel, no provision is made for passing its flood waters, and when floods occur they inundate a considerable area of country surrounding the canal on both sides, and necessitate upon their subsidence the reconstruction of quite a portion of the canal banks.

In the 12th mile from the head is a flume 1 200 feet long spanning Kramer Creek. This flume rests on oak pile bents capped with heavy timber, and is well constructed and braced.

In addition to the 41 miles of main canal now existing, there are two main feeder branches which were constructed independently of the Arizona Canal, parallel to its course, and lying at distances of a few miles below it down the slope of country between it and the Salt River. The Arizona Canal Company have, however, purchased these, and use them as a portion of their system, the main Arizona Canal acting as a high line feeder to these auxiliary canals. At about the 20th mile of its course the cross-cut feeder is diverted from the main canal by means of two substantial sets of regulating gates, one of six openings in the main canal and the other of five openings at the head of the cross-cut. These regulators are founded on sheet piling and are constructed of wood, the gates being operated from an overhead bridge by means of hand-levers. The total length of the cross-cut is about 4 miles, the bottom width 22 feet, slopes 1½ to 1, grade 2.65 feet per mile, and capacity 375 second-feet. As it is built straight across the country down its slope, the total fall in its 4 miles is 128 feet. This drop is compensated by twenty-eight falls averaging about 5 feet in height each, and substantially constructed of wood founded on sheet piling. The checks in these falls (see Plate XXX) consist of flash-boards let in between posts, below which is an apron of wood 12 to 16 feet in length, while the banks of the canal are well protected by wings and sheet piling. The first parallel feeder canal supplied by this cross-cut is the Grand Canal, below which the dimensions of the cross-cut are slightly diminished until its terminus is reached at the second feeder, the Consolidated Ditch. The Grand Canal has a bottom width of 20 feet, a depth of water of 3½ feet, and a fall of 2 feet to the mile, giving a velocity of 3 feet per second and a capacity of 210 second-feet. Its side slopes are 1 on 1½ and it is constructed about three-fourths in excavation. The dimensions of the Grand Canal are a little greater than those of the Consolidated, which latter is bifurcated just below its head into two main branches, the Maricopa and Salt River Valley Canals.

The main Arizona Canal has about 125 miles of laterals constructed, which command 77 000 acres of excellent arable land. The Consolidated and Grand Canals are together about 70 miles in length and have 75 miles of laterals, covering 73 000 additional acres of land. It is difficult to ascer-



FALL OR CROSS-CUT ON THE ARIZONA CANAL.

tain the cost of the Consolidated properties, as they were constructed by various independent owners and by farmers. The main Arizona Canal, however, exclusive of these branches and all laterals, but including the dam and other works on its line, cost $580 000, the laterals $25 000 more, and the improvements to the present dam an additional sum of $25 000, making its total original cost about $630 000. As an indication of the value of land in this region when supplied with water for purposes of irrigation, it may be stated that water rights are sold to owners of land at the rate of about $1 000 for 80 acres, in addition to which an annual rental of $1.25 per acre irrigated is charged. On a portion of the line, water rentals are from $50 to $100 per second-foot.

As an example of the difficulties encountered and overcome in the present and last stage of irrigation development in this country, a brief description will be given of the canals of the Wyoming Development Company. These canals are now under construction and are partly completed. They cannot be considered good examples of the latest engineering practice, as no engineer is employed in building them, though originally designed by one. These canals, however, show the ingenious methods employed and the distances overcome in transporting water to valuable irrigable lands.

The canal is diverted from the Big Laramie River at a point about 120 miles north of Cheyenne and commands approximately 60 000 acres of irrigable lands between Sybille and Chugwater Creeks. At the point of diversion the Big Laramie has a maximum discharge in times of flood of about 8 000 second-feet, while the minimum discharge is as low as 100 second-feet at the lowest stage of summer flow. The water is diverted from the Laramie by means of a crib and stone dam 150 feet in length, resting on the coarse bowlder and gravel bed of the river. The maximum height of the weir is 4 feet, its top width being 16 feet, and it has a 4-foot vertical drop on the down stream side to an apron 16 feet in length composed of logs laid parallel to the course of the stream. The canal heads at the south end of the weir, and is regulated by a series of six wooden gates, each 5 feet wide and 9 feet in height, and operated by lever and ratchet. The first 2 000 feet of the canal consists of an open canal and approach cut in gravel, having a 30-foot bed-width, and capable of carrying 8 feet depth of water. Its grade is 7 feet per mile, and its side slopes $\frac{1}{2}$ to 1, the discharge capacity being 650 second-feet. This

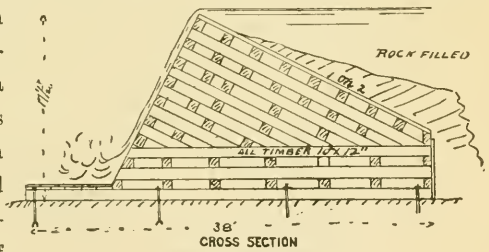
open cut terminates in a tunnel carried through a portion of the divide which separates the Big Laramie from Blue Grass River and tailing into a short branch of the latter stream. The tunnel is in rock, unlined, with an approximate cross-section of 7 x 8 feet, and is 31 feet in length with grade varying from 50 to 100 feet per mile. The terminus cut at the exit of the tunnel is in rock, and is 25 feet in maximum depth and 1 000 feet in length, the fall being 450 feet per mile and the bottom width 8 feet.

After being discharged into the Blue Grass Creek the water flows down this stream for a distance of 13 miles to its junction with the Sybille Creek, down which it flows a further distance of 1½ miles to a second diversion weir constructed across that stream. In Blue Grass Creek the total fall is 1 700 feet over a rock bottom and through a narrow and tortuous channel, and in Sybille Creek it falls an additional 30 feet. The diverting weir in Sybille Creek is constructed of wood founded on piling driven in the river gravels. The form of this weir is similar to the open weir described for the Del Norte Canal, and consists of eight bents 8 feet long across the stream and 8 feet in height, closed by flash-boards presenting a vertical face up-stream. The two banks of the river are protected by wooden planking for a distance of 12 feet up-stream and 18 feet down-stream. The second length of the canal heads above this weir, and is controlled by a set of regulating gates of the same dimensions and general form of construction as those described for the first section of the canal, but having a total width of 25 feet, disposed in five gates each 5 feet between centers, and built as a continuation of the diverting weir. This second section of the canal, which is the main distributing canal proper, is 34 miles in length, and is constructed for the first 20 miles of its course in gravel and hardpan, with much difficult side-hill work, some cuts being as deep as 40 feet. Its bed width is 25 feet, slopes 1 to 1, depth of water 5 feet and grade 2 feet per mile. In its course it encounters considerable side hill drainage, all of which is let into the canal by simple inlets, while waste-ways or escapes are constructed in the opposite bank of the canal and closed by simple flash-board regulators with discharging capacities according to circumstances, up to an extreme width of 30 feet. A few miles below the head of this main canal, an upper lateral is diverted which is controlled by a simple flash-board check built in the main canal. About 12 miles below the head of the main canal a second main distributing canal is diverted from Sybille

Creek by means of a weir similar to that just described, the height of which is 5 feet and its length 40 feet. This canal is 20 feet wide at bottom, and it has 4½ feet depth of water, slopes 1 to 1, and a grade of 2 feet per mile. It is 20 miles long and encounters some very heavy side-hill work in the first few miles.

A fine example of the present stage of canal development, and one which has been designed and constructed under the supervision of some of the best of our irrigating engineers, is the Bear River Canal in Utah, which is diverted from the Bear River at a point about 3½ miles above Colliston. This system consists essentially of a main western and main eastern canal, diverted one from either bank, while the former is divided into two principal branches, from which are taken the various laterals supplying the private ditches. At present the diversion weir and the first 6 to 8 miles of heavy rock work in the cañon are completed, and bring the water to the level of the broad valley lands which it is intended to irrigate. In all there are now completed 60 miles of main canals, with all flumes, regulators and other works, and a few miles of distributaries. When finally completed there will be 150 miles of main line and principal distributaries, and a sufficient mileage of laterals to enable the canal to irrigate 200 000 acres of excellent land.

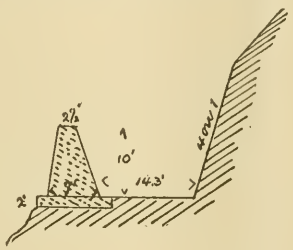
The average minimum discharge of Bear River at the weir site is about 1 000 second-feet, occurring in midsummer, while the maximum recorded flood discharge is less than 9 000 second-feet. The diversion weir, of which a cross-section is shown, is admirably located between high rock abutments, and with shallow rock foundation. It is constructed of crib work filled in with earth and loose stone, having a total height of 17½ feet, and a length at the bottom parallel to the channel of the stream of 38 feet, its length between abutments being 370 feet on the crest. The up-stream slope is 1 on 2 and the down-stream slope about 1 on ½, the water falling on a wooden apron, anchored to the bed rock. All timber is 10 x 12 inches, and is drift-bolted to the rock bed. The head-gates on the west side consist of five gates, each 4 feet wide by 7 feet high, of iron,



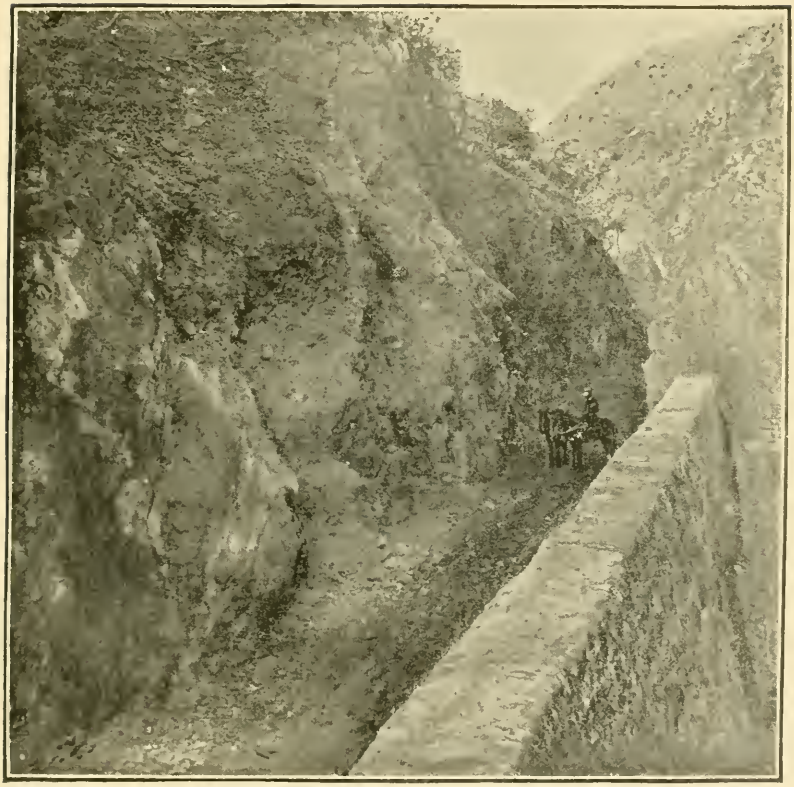
CROSS SECTION
BEAR RIVER WEIR

and built into substantial masonry-in-cement abutments and piers founded on rock. The head-gates to the eastern side canal are of essentially similar form of construction.

The first 2 miles of the east side canal are in heavy rock work, in which are two tunnels about 14 x 14 feet cross-section in homogeneous rock and unlined, the first being 423 feet long, and the second 200 feet in length. In this portion of the canal are a number of deep rock cuts (see Sketch and Plate XXXI), the greatest of which is 96 feet in height on the upper side, while in numerous places the lower side of the canal consists of a built up retaining wall of rubble-in-cement masonry, usually 10 feet in height inside, with a top width of $2\frac{1}{2}$ feet, and a width at grade line of $7\frac{1}{2}$ feet, as shown in the accompanying diagram. For 5 miles below the rock work the canal is excavated in steep earth hillsides, the slope of which is about 3 to 1. In this portion of the line are several deep fills across ravines, the beds of which are drained by means of drainage culverts carried through the bank. The greatest fill is 108 feet in depth at the center, and 500 feet long on grade, on the top of which is laid a wooden flume which carries the canal water and rests on long piling driven 60 feet into the fill. There are several such fills as this on both of the main canals, and two or three of them have already been washed out owing to leakage from the flumes or insufficient drainage way through the embankment. After reaching the level country the east side canal has a bottom width of 50 feet, a depth of water of 7 feet, with side slopes of 1 to 1 and a grade of 1 foot per mile. This canal as projected will have a length of 50 miles, terminating at the Ogden River, in the City of Ogden.



The west side canal for the first 9 000 feet of its length is likewise constructed in heavy rock cañon work. In this portion of its line are six tunnels, varying in length from 57 to 279 feet, and having the same cross-section and grade as the east side tunnels. Below and between the tunnels are eleven big retaining walls, of rubble masonry, having dimensions similar to those on the east side canal; while this portion of the canal has a bed width of 14.3 feet, a depth of 10 feet, and nearly vertical slopes through the rock work. Twelve hundred feet below the head of this canal is an escape gate, and 600 feet further down is a second



VIEW ON BEAR RIVER CANAL.

PLATE XXXII
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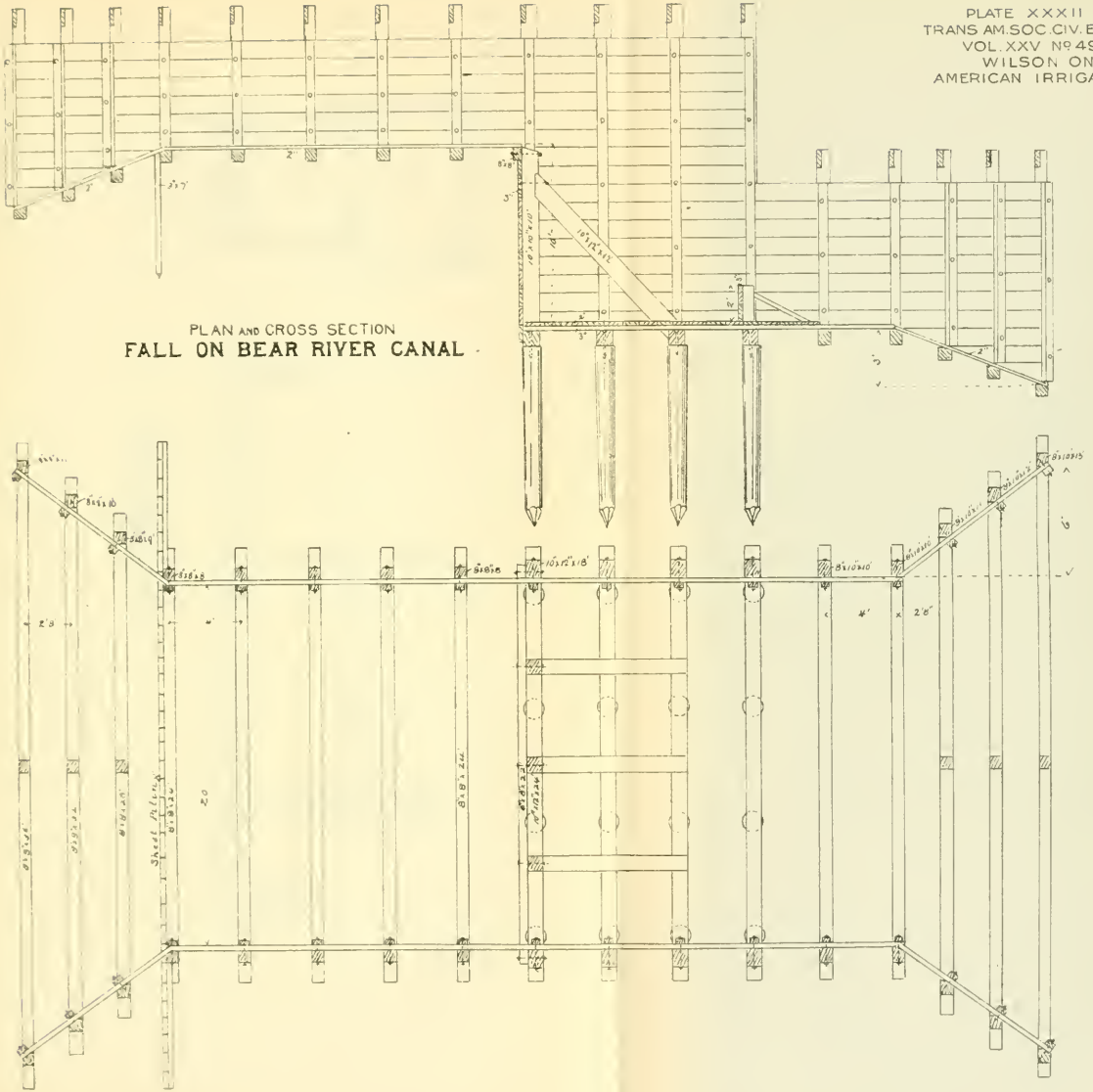

PLAN AND CROSS SECTION
FALL ON BEAR RIVER CANAL

PLATE XXXIII.
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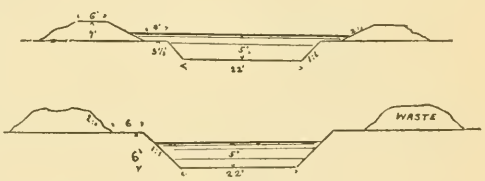
AMERICAN IRRIGATION

IRON AQUEDUCT
ACROSS MALAD RIVER
BEAR RIVER CANAL

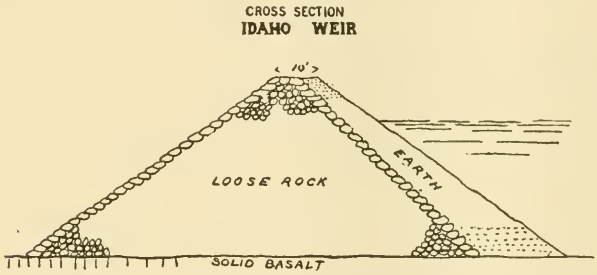
escape gate, both discharging back into Bear River, and both having clear discharge openings of 12 feet in width, closed by three wooden gates, sliding between iron posts let into masonry piers. Below the second escape is a regulator closed by five gates, each 4 feet in width, by which the discharge in the canal itself can be controlled. After leaving the rock work, the canal enters steep hillside excavations in earth and clay similar to that on the east side, and extending to the sixth mile. In this side-hill work the bottom width of the canal is 14.3 feet, depth 12 feet, side slopes approximately 1 to 1, while there are several deep fills and cuts similar to those described for the east side. Beyond the ninth mile the canal crosses Malad River and valley on a high iron bridge and flume, 378 feet in length, and 80 feet in maximum height, supported on iron trestles, the river span of which is 70 feet. This bridge has approaches by means of wooden flumes which are 500 feet long, 20 feet wide in the clear, and carrying 7 feet in depth of water, the waterway over the iron bridge consisting of a similar wooden flume. In its course, the main western line has three falls, each of 7 feet in height, to lose grade, and it is projected for a length of 40 miles terminating at the Great Salt Lake.

Six miles below the head of the West Side Canal the Corinne branch is diverted, which terminates in the twentieth mile of its course at the city of that name. This branch at its head is 22 feet wide at the bottom, carries 5 feet depth of water (see diagram), and is controlled at its head by a double set of regulating gates. In the main and Corinne branches, the bottom and sides of the canal are well protected by wooden aprons and wings. As this branch runs down the slope of the country it has in its course sixteen vertical falls varying from 4 to 12 feet in height, one of which is shown in Plate XXXII. At the fourteenth mile, Malad River is crossed on a high iron bridge founded on piles and iron cylinders filled with concrete. This bridge (see Plate XXXIII) consists of three principal bents from 25 to 60 feet in length, the peculiarity of its construction being that the superstructure forming the bridge itself is of iron plate girders and constitutes the flume which carries the water. The chief trouble with this iron flume is



the difficulty of keeping its junction with the wooden flume approaches in good condition. Among other works on the line of this canal are some inverted siphons or culverts of wood, carrying branch canals under the larger canals. One of these siphons consists of two tunnels of 8 x 8 timber, each having a clear waterway of $5\frac{3}{4}$ x $2\frac{1}{3}$ feet.

A magnificent illustration of the modern development of canal construction in the Western States is the canal of the Idaho Mining and Irrigation Company taken from the Boise River about 12 miles above the City of Boise, Idaho. This canal is interesting, not because it encounters any serious difficulties in construction, but because of the careful and intelligent thought which has been displayed by the engineer, Mr. A. D. Foote, in every detail of its design, and because of the magnitude of the canal and the permanent character of the various works constructed upon it. This canal is built with the two-fold object of irrigating about 350 000 acres of excellent agricultural land which is situated



between the junctions of the Boise and Snake Rivers, and of supplying water for the hydraulic mining of extensive placer deposits on the lower banks of the Snake River, a short distance above its junction with the Boise. The climate and soil in this region are such that abundant crops of the more valuable grains, vegetables, and such fruits as apples, peaches, pears, grapes, etc., can be cultivated with the aid of irrigation. The maximum average discharge of the Boise River occurs between the months of April and July and is about 10 000 second-feet, increasing to the greatest known flood discharge of about 30 000 second-feet. Its minimum discharge is seldom less than 1 200 second-feet and the river may fall as low as this during the remaining months of the year. If necessary to supplement its discharge by artificial storage of its waters, several excellent reservoir sites have been discovered upon its headwaters, the combined capacity of which is nearly 225 000 acre-feet; while

PLATE XXXIV.
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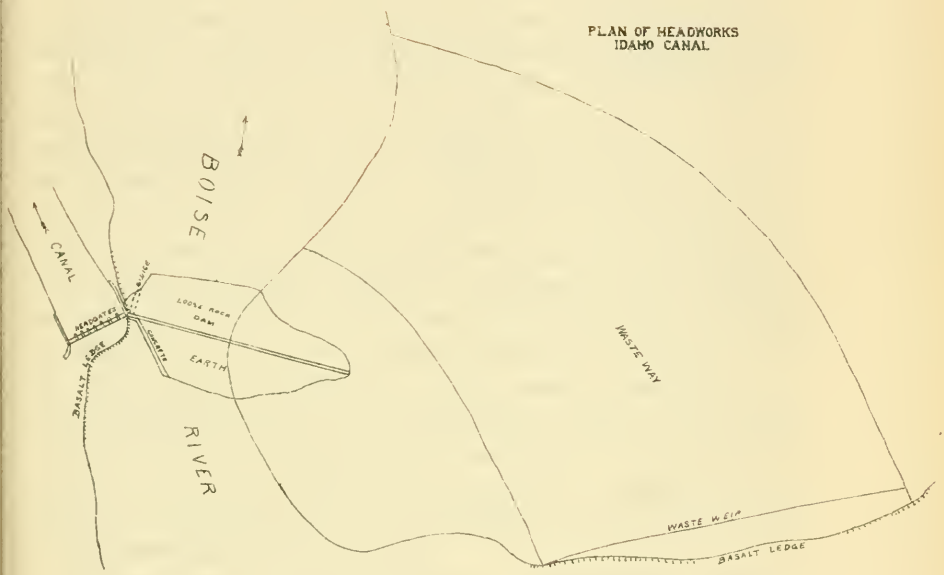


the estimated cost of constructing these, averages about $1.50 per acre-foot, an unusually low figure.

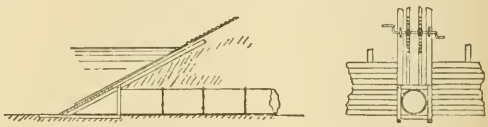
The diversion dam at the head of this canal is particularly well located, and as shown in the cross-section and Plate XXXIV consists of two parts, a main dam of loose rock with earth filling on the upper side crossing the channel of the Boise River, and over which it is not designed that the flood-waters shall pass; and a waste-way excavated through gravel on the north side of the river at a point about 345 feet above the dam, while on this waste-way is built a masonry weir over which flood-waters are discharged. The dam is 220 feet long on its crest and 43 feet in height above the river bed, with a top width of 10 feet, of which 3 feet of the inner side consists of the top of the earth filling, which at the bottom of the dam is 20 feet in width. The outside or rock slope is $\frac{1}{2}$ to 1 and the up-stream or earth slope $1\frac{1}{2}$ to 1. The whole is founded on the solid basalt which outcrops across the entire channel of the river at this point. Just above the dam a basalt ledge 12 feet in height borders the river bank, and on this ledge the waste-weir is constructed with a length of about 500 feet. The crest of this waste-weir is 8 feet below the crest of the dam, and the length of the waste-way is 200 feet from its entrance to its discharge point below the dam, while its capacity is estimated to be at least 30 000 second-feet. The cross-section of the weir is peculiar, being about 19 feet in width parallel to the length of the waste channel and 8 feet in maximum height; its upper slope has a batter of 6 to 1, while its lower slope has an easy ogee-shaped curve similar to that which the flowing water will assume.

The canal heads on the south bank of the river immediately adjacent to the end of the dam, in which latter is constructed an under or scouring sluice, the object of which is to prevent the deposition of silt in front of the head gates of the canal. These gates rest on the summit of a similar basalt ledge to that described as bordering the northern bank of the river and of equal height. The gates are constructed of the best rubble-in-cement masonry and are covered on top, forming an over-head bridge 19 feet in width. The regulating gates are 8 feet wide in the clear, and 19 feet in height to the crown of the semi-circular arch. The total height of these head-gates above the basalt ledge is 21 feet, and their height above the river bed 33 feet. The head-gates will be closed by means of roller curtains made of steel plates and angle iron for a height of 10 feet above the sill, above which the slats of the curtain are made of

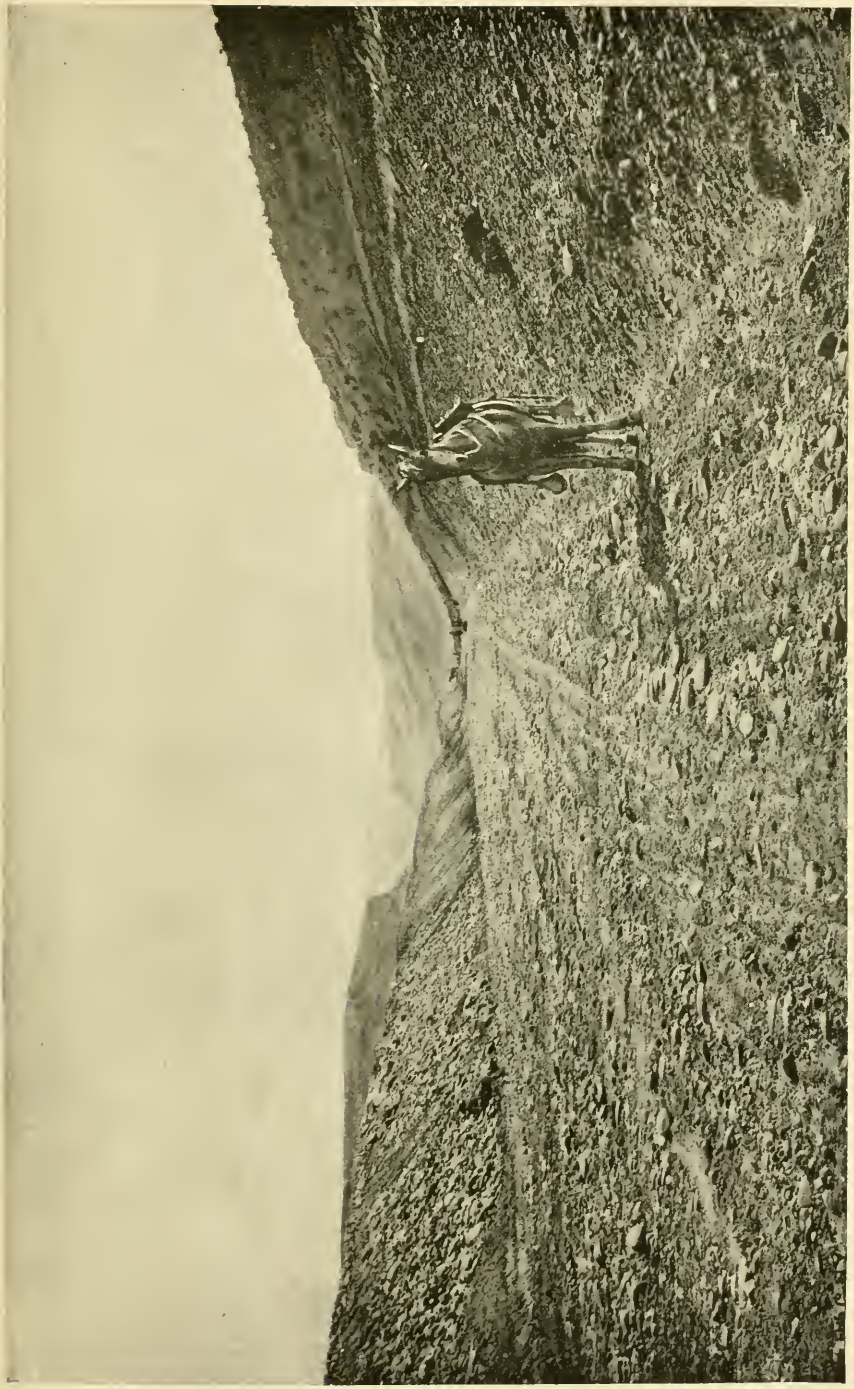
pine wood, each slat 8 inches wide. Each curtain is composed of twenty steel and eight wooden slats fastened to a central cast-iron roller at the bottom, and they are wound up by means of a single chain operated from a winch on the over-head bridge. These gates are modeled somewhat after those used on the open weirs on the River Seine, in France.

The first 2 miles of the main canal are excavated in loose lava rock in the steeply sloping sides of the cañon. There are no bad cuts nor fills encountered in this cañon work, and below it the canal encounters a couple of rather steep gravelly bluffs which its grade has to surmount, before it reaches the main body of the irrigable land. These lands are reached in about the eighth of a mile, after which the work is in simple level earth excavation, and encounters no other difficulties than three short stream crossings; one at the 15th mile, where Five Mile Creek is crossed by means of a short terreplein or raised embankment, with a wooden flume of about 24 feet in length founded on piles across the channel of the stream, and two other similar crossings at Ten Mile and Indian Creeks. In the first 8 miles of gravel bluff work below the cañon, the canal is constructed one-half in excavation and one-half in fill, the lower side being a high embankment which was constructed with scrapers and well tramped over. In one place this embankment is over 40 feet in height for a length of about 1 000 feet and is practically an earth dam for this distance. The capacity of the main canal (see Plate XXXV) is estimated at 2 585 second-feet; its bed width is 40 feet; height of bank, 12½ feet; depth of water 10 feet, and velocity of current, owing to its construction in the gravel and rock, as high as 5 feet per second. After getting above the bluffs to the level open country, all slopes to banks, both in and outside, are 1 on 1½, and a uniform grade of 2 feet per mile is given. The main canal has a total length of 70 miles, and tails into the Snake River at the placer mines above mentioned. At the 34th mile of its course the main eastern branch is diverted, which is 15 miles in length and likewise terminates in the Snake River.

CROSS SECTION AND ELEVATION
IDAHO CANAL REGULATOR HEAD



On the main line at the end of the cañon is an escape for the discharge of surplus water which tails back into the Boise River. This escape consists of a number of 48-inch cement pipes laid through



MAIN LINE OF THE IDAHO CANAL.

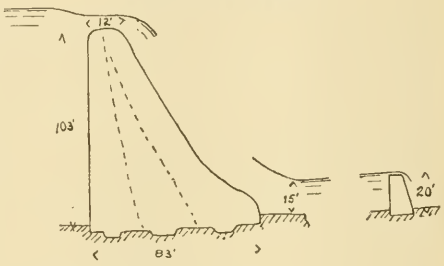
the canal bank and closed on the inner or water face by a sloping wooden gate laid at the angle of the canal banks and operated from above by rack and pinion. All distributary heads are constructed in the same manner of concrete pipes laid through the canal bank. In addition to the first escape below the cañon, provision is made for wasting water from the flumes crossing Five Mile, Ten Mile and Indian Creeks into those creeks. On the main line just below the bifurcation of the eastern branch is a fall 49 feet in height; about 6 miles further on is a 12-foot fall or drop, and further on, on the same line, are a 22-foot and 28-foot fall. These are not true falls, but are chutes or rapids constructed at such points as to take advantage of rock which crops out near the surface, and in which they are excavated, other portions of the rapids being masonry lined. All similar drops on the minor branches consist of inclined wooden flumes or rapids, laid on slopes of from 1 to 5 feet in 100.

By far the most interesting and magnificent canal under construction in this country is that of the Turlock Irrigation district on the west side of the San Joaquin Valley, in Central California. This canal is interesting as an example of the result and operation of the Wright Irrigation Law, recently passed in California, as well as from the magnificence of its works and the skill displayed by its engineer, Mr. E. H. Barton, in overcoming the difficulties encountered in locating the first few miles of its diverson line. The Turlock district is so outlined that all of the 176 000 acres included within its borders can be watered from the same source of supply. This district is bonded for $600 000, but it is estimated that before the works under construction are completed a further sum about equal to this will have to be raised by the issuance of additional bonds.

The Turlock Canal is diverted from the Tuolumne River at the mouth of the cañon. This river may attain a maximum flood discharge of 150 000 second-feet, while its minimum discharge rarely falls below 2 000 second-feet. The head-works of the canal consist of a masonry dam, which is constructed as a common diversion weir for the Turlock Canal and the canal of the Modesto Irrigation district, which heads on the opposite or north bank of the river. This weir is located between high cañon walls 2 miles above the town of La Grange, at a point where the abutments and foundation of the weir are on a firm homogeneous dioritic

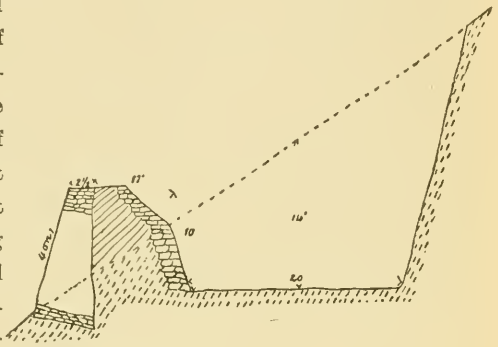
basalt in which scarcely any excavation will be required. The weir, over which the flood discharges of the river will pass, is constructed throughout of uncoursed rubble masonry in Portland cement, and has a length on its crest of 310 feet. Its maximum height, including foundations, is 103 feet, and the maximum height of the overfall of water is 98 feet, to a water-cushion formed by a subsidiary weir built of rubble masonry and located at a point about 200 feet below the main weir in a narrower portion of the cañon, as shown in the accompanying diagram. The main weir has a top width of 20 feet, and is designed on a cross-section somewhat similar to that of the Vyrnwy Dam for the Liverpool water supply; its upper face being nearly vertical, while its lower face is given a curve similar to that which the water flowing over its face will assume. This weir is so designed that the maximum pressure on the toe, when full, will be 6.3 tons per square foot, which is well within the limits which the material will stand. The subsidiary weir is 120 feet long on top, 12 feet in width and 20 feet in maximum height, and will back the water up to a depth of 15 feet on the toe of the upper or main weir giving a water-cushion of that depth into which the flood-waters will fall, thus to a certain extent diminishing their shock.

CROSS SECTION
TURLOCK WEIR



The arrangement of the head-gates of the canal is peculiar. The canal is diverted from the south bank of the cañon at a point about 50 feet above the end of the main weir. Owing to the great floods which occur in this narrow cañon, the water may rise as much as 15 feet in an hour, and the maximum height which it is estimated to reach above the sill of the canal is 16 feet. The pressure of this height of water on the regulator head would be so great as to greatly increase the cost of its construction, accordingly the canal heads immediately in a tunnel 560 feet in length, blasted through the rock of the cañon walls and having no regulating apparatus at its entrance. Where it discharges into the open cut, which is the commencement of the canal, regulating gates and scouring or escape sluices are placed. The entrance tunnel is 12 feet wide at the bottom, 5 feet in height to the springing of the arch, above which it is

semi-circular with a 6-foot radius. Its slope is 24 feet per mile, and it is excavated in a firm dioritic rock which requires no lining. The regulators in the canal head below the exit of the tunnel consist of six gates, each 3 feet wide in the clear and 12 feet in height. These gates are constructed of timber and iron, and slide on angle iron bearings firmly let into the rock and set in concrete. The escape is set at right angles to the canal line, heading immediately above the regulator between it and the end of the tunnel, and tailing back into the Tuolumne River a short distance below the subsidiary weir. Like the regulator, the escape consists of six gates, each 3 feet wide in the clear, 12 feet high, and constructed of similar material and in like manner. It is estimated that, whereas a maximum flood of 16 feet over the sill of the tunnel will give a discharge in front of the regulator and escape of about 4 000 second-feet with a velocity of 20 feet per second, the wasting capacity of the escape will be at least 6 000 second-feet, thus fully insuring the canal against accident from this source.



Below the regulating gates the main canal proper begins, having a capacity of 1 500 second-feet. For the first 6 200 feet it is excavated in slate rock on a steep hill-side. It has a bed width of 20 feet, depth of water of 10 feet, and an upper rock slope of $\frac{1}{2}$ to 1; while the lower bank or down-hill slope, where gullies are crossed, is, as shown in the accompanying diagram, built up with an inner slope of $\frac{1}{2}$ to 1, and faced with 18 inches of dry laid retaining wall inside and outside, the interior of the bank consisting of a well puddled earth core 12 feet in width. Where this portion of the canal is on ordinary sloping ground, not crossing gulches, its dimensions are the same, but the inner face only has the 18 inches of rip-rapping, and the down-hill slope of the bank consists of dirt and other spoil, the top width of the bank in such places being 5 feet, and the puddle wall 5 feet in thickness. This portion of the canal line has a grade of 7.92 feet per mile, which gives a velocity of $7\frac{1}{3}$ feet per second.

At the end of this slate-rock work the canal empties into Snake

Ravine, just above its junction with the Tuolumne River, up which ravine the water of the canal runs for 940 feet. This is effected by constructing an earth dam across the mouth of the ravine just below the entrance of the canal, which raises the surface of the water so as to form a small settling reservoir, and produces a flow up the course of the ravine for the distance above mentioned. The earth dam is 20 feet wide on top, 318 feet long on the crest, has slopes of 2 to 1 and a maximum height of 52 feet. This dam was partly constructed of material borrowed from its abutments and the canal excavation, and partly by a silting process from material washed out of a hydraulic cut at the upper end of the ravine. This hydraulic cut is 800 feet in length and 45 feet in maximum height, with slopes of 1 to 1, and a grade of 5 feet per mile. Owing to the abundance of water procurable, this cut was more cheaply excavated by the hydraulic mining process than it could have been by other means. At the far end of this cut the canal enters an old hydraulic wash, in which its waters spread out, and which is utilized as its channel for a length of 2 380 feet, after which it enters a rock cut 860 feet long, with a maximum depth of 45 feet and a similar cross-section to the cut first described.

At the end of this rock-cut the canal water is discharged into Dry Creek, down which it flows for a distance of 6 500 feet on a grade of 12 feet to the mile, and from which it is diverted by means of an earth dam 460 feet long. This dam has a maximum height of 23 feet with both slopes of 3 to 1, and is rip-rapped to a depth of 3 feet on its upper face. At the south end of the dam it abuts on sandstone rock, in which a waste-way is cut 50 feet wide, with its sill 4 feet below the crest of the dam, and which will discharge back into the creek 180 feet below the toe of the dam. Between the waste-way and the end of the dam is a waste-gate which it is intended shall be used in times of freshets, for Dry Creek has a maximum discharge of 4 000 second-feet, and as the freshets are quick and violent a large wasting capacity is necessary. These waste-gates are ten in number, each 3 feet wide in the clear and 10 feet in depth. They fall automatically outward or down-stream, being hinged at the bottom to a concrete floor laid on the bed-rock, and when raised they are attached by chains to the piers.

From Dry Creek the canal is excavated for about a mile in heavy, sandy loam, in which it has a bed width of 30 feet, with slopes, 2 to 1, a depth of 10 feet, and a grade of 1½ feet per mile. At the end of this

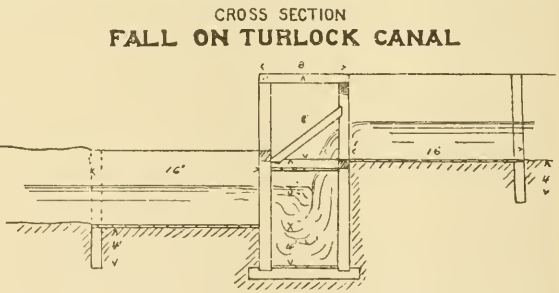
TUNNEL ON DRY CREEK.

excavation the canal crosses Dry Creek on a flume 62 feet in height and 450 feet long, after crossing which it enters a series of three tunnels, the cross-sections of which are nearly similar to that of the first tunnel described, while they are excavated in a tufa and sandstone which will require no timbering. The first tunnel (see Plate XXXVI) is 211 feet in length, the second 400 feet, and the third 400 feet in length, while they are separated by short, open cuts, excavated in hardpan and clay, which are respectively 250 and 300 feet in length. The last tunnel discharges into Delaney Gulch, which is crossed directly by constructing a high bank or earth dam below the canal, the total length of which is 180 feet, its maximum height being 40 feet, and its top width 20 feet. The volume of discharge of this gulch is so trifling that it was unnecessary to provide a waste-way or escape at this point. Immediately after crossing the gulch the canal enters a cut 8 feet in maximum depth, with the same cross-section and grade as the first cut, and having a length of 3 300 feet. The canal is now widened to a bed width of 35 feet and depth of 10 feet, and is given a grade of 1 foot per mile. At the end of a mile and a half, Peasley Creek is crossed on a trestle and flume 60 feet in height and 360 feet long, the water-way on which is 20 feet wide and 7 feet in depth. This flume is provided with an escape constructed in its bottom and discharging into two small sloping flumes which lead the water down into the bed of Peasley Creek.

At the end of the flume the main canal is reached and traversed for a distance of 11 miles, in which course are two rock cuts, each 3 000 feet long, and respectively 20 and 30 feet in maximum height. In these cuts the cross-section is 35 feet wide on bottom, depth of water 7½ feet, and grade 5 feet per mile. The remainder of this length of the canal varies in cross-section according to soil, but the majority of it has a bottom width of 70 feet, and depth of water of 7½ feet, slopes 2 to 1, and a grade of 1 foot per mile. Previous to choosing this location a careful detailed topographic survey was made of the entire Turlock district at a scale of 1 000 feet to the inch and in 5 foot contours, and from this map the location was chosen. This is so designed as to take advantage of several natural channels consisting of the creeks and gulches mentioned, of an old hydraulic excavation, and of the opportunities afforded for excavating two deep cuts by the hydraulic process. To be sure, one slight disadvantage may be claimed, because of the large water-surface exposed to evaporation by the creation of the various small reservoirs. This,

however, is not a defect, for the water supply will exceed the demands for a number of years to come, and these basins act as settling reservoirs in which the turbid water of the river will be freed of much of the sediment which it carries in suspension. Before the water becomes scarce, these basins will be filled full by sedimentation, and in the flats thus created a canal channel of proper cross-section can be maintained. It is expected that in the course of a few years the river bed above the main diversion weir will be silted up to the level of its crest; there are no objections to this, as it is not intended to utilize the basin above the weir as a storage reservoir.

The main canal as outlined above consists for the 18 miles of its length of a purely diversion channel, the object of which is to bring the water to the irrigable lands included within the area of the Turlock district. At the terminus of this diversion line the canal will at once begin to do duty by watering the lands, and below this point the main line is divided into four main branches, each of which has a bottom width of 30 feet, depth of water of 5 feet, and grade of 2 feet per mile, their aggregate length being 80 miles. In addition to these main branches minor distributaries leading the water to each section of land and having a total length of 180 miles are being constructed. The lower discharge branches and distributaries are so designed as to give a uniform velocity of 2½ feet per second, in order that any matter carried in suspension will be held up until deposited on the agricultural lands instead of in the canals.



It is interesting to note that while the contract prices for excavating the two deep cuts by ordinary processes was forty-eight cents per cubic yard, they were excavated by the hydraulic process for thirty-one cents per yard, the material being moved one-half mile, while gold to the value of $13 000 was saved in the washing, thus reducing the cost of excavation by four cents per yard. In the course of the main canal nine falls are inserted in order to reduce the grade. These vary in height

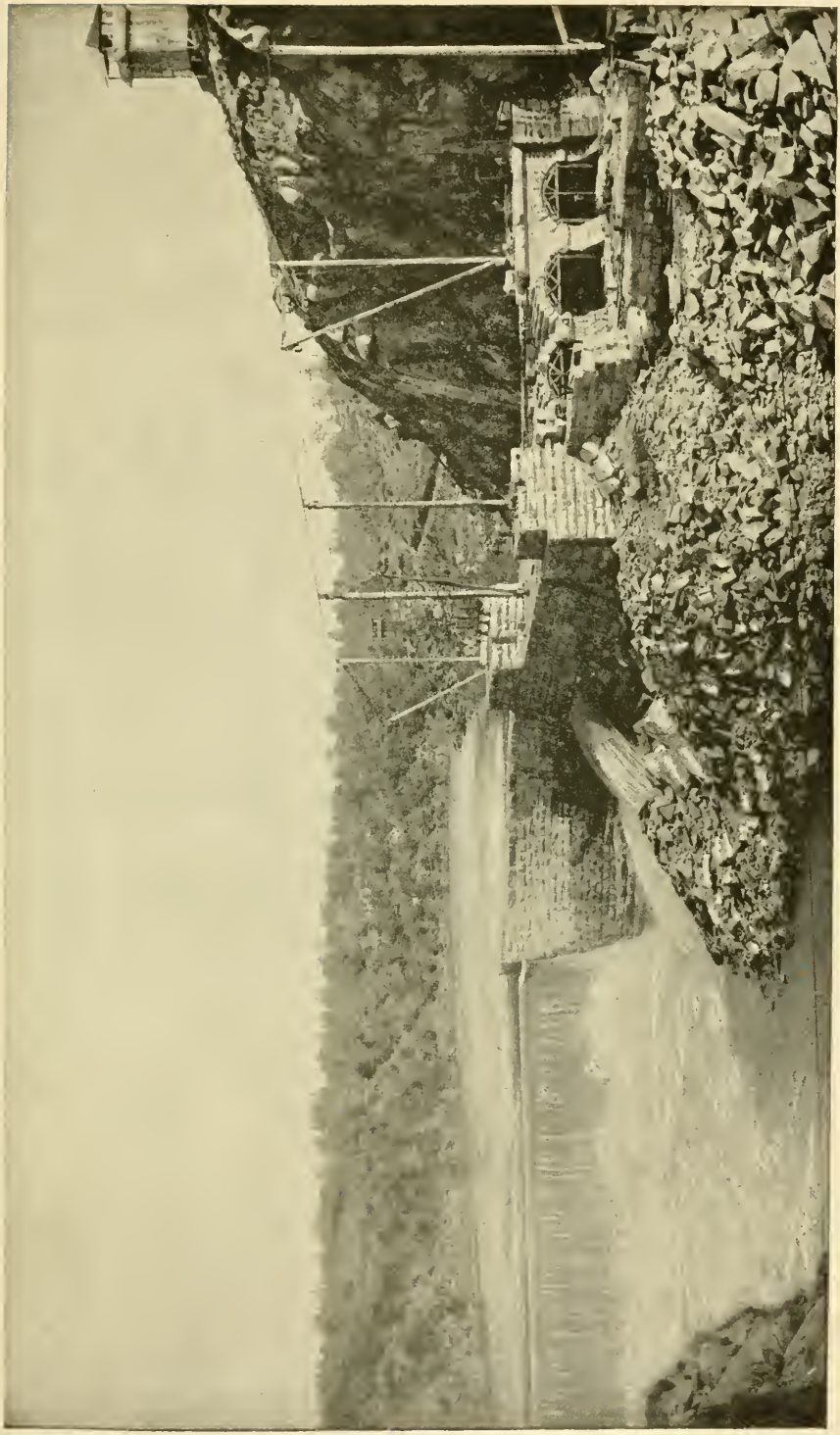
from 4 to 11 feet, while the bed width of the channel is contracted from 70 feet to a clear width of 40 feet through the falls, the general form of construction of which is indicated in the accompanying illustration. Falls 5 feet in height drop into water cushions 4 feet in depth, the 11-foot falls having water cushions 6 feet in depth, the object of which is to break the force of the fall. Across the channel the falls are divided into four bays, each 10 feet wide, by means of vertical rows of plank inserted in the lower portion of the fall. These diaphragms are intended to keep the current straight after passing the fall, thus preventing back currents and scouring, in which effect they are aided by the widening of the channel again by means of wings from 40 to 70 feet. It was calculated by the weir formula that by narrowing the channel above the fall from 70 to 40 feet the lowering of the depth of the water would be prevented with its consequent increase of velocity and scour.

The works of the Folsom Water Power Company, situated on both banks of the American River, near Folsom, Cal., are the most substantial and elaborate of their kind that have been constructed either in this country, Europe, or India, so far as the writer's experience indicates. These works are built with the object of furnishing water power to the State Prison at Folsom, and for manufacturing and milling purposes in the City of Folsom, after which the water used in producing the power passes on to the irrigable lands which the canals will develop in the near future. The chief feature of interest in connection with these works as at present constructed is the massive and substantial character of the diversion weir which is situated on the American River, 1½ miles above the town of Folsom, and of the main canal which is completed as far as Folsom. The waters carried in the main east side canal after they have been utilized at the second fall in Folsom for the development of water power, can be readily transported in simply constructed branches and laterals to 125 000 acres of excellent irrigable lands in Sacramento County, and a ready sale will be found for all the water which can be made available for this purpose. One of the peculiarities of this project is the arrangement by which it is constructed in co-operation with the State Prison, which is situated near the site of the dam a mile above Folsom. In consideration of the Water Power Company giving the prison authorities about 400 acres of quarry land adjacent to the prison, a right of way through the company's property for a railway, and a 7-foot head of fall

in the prison yard for the generation of water power from the entire discharge of the canal, the prison authorities have given to the company all the labor required in the construction of this work, this labor being performed by the convicts in the prison.

The great diversion weir (see Plate XXXVII and XXXVIII) was designed and built under the direction of Mr. P. A. Humbert, chief engineer. This work is constructed of the best coursed granite rubble-in-cement, founded on a firm bedrock and abutting against granite walls. The main tangent of the weir across the channel of the American River is 250 feet in length, while the first 250 feet of the canal on the left bank of the river is constructed as a prolongation of the weir, thus giving 500 feet in length of discharge over which the floods of the American River can fall. The flood discharges of the stream are very violent, and it is anticipated that the maximum flood passing over the crest of the weir will be about 34 feet in depth, a depth of 31 feet having already been gauged. As will be observed from the diagram, the cross-section of the weir is very substantial; it is 24 feet in width on top, 87 feet wide at the bottom, with a massive buttress extending well down below the toe, the object of which is to counteract the scouring effect of the water. It is 69½ feet in height on the up-stream face, its total height being 98 feet. In the center of the weir, and on a level with the low-water surface of the river, is a set of three under or scouring sluices, each 4 x 4 feet, which discharge the silt-laden flood waters of the river under a head of 60 feet. While these under-sluices have not impaired the integrity of the structure, they have been of little service in preventing the deposit of silt, as their area compared with that of the flood is relatively small.

One hundred and eighty feet in length of the center of the weir is lowered to a depth of 6 feet below that of the remainder of the crest, and forms a discharge weir which is closed by a shutter the height of the opening. As this river carries in suspension a great amount of sediment, it was expected that without some flushing apparatus it would soon silt up on a level with the head-gates, accordingly in order that the silt deposit in this place may be kept down to a level with the sills of the head-gates, this waste-way has been introduced. When the shutter closing the waste-way is lowered the water will carry out the sediment down to a level with the crest of the waste-way, and when raised it will divert the water into the head of the canal. This shutter is of peculiar construction:



HEAD-WORKS AND WEIRS ACROSS AMERICAN RIVER, FOLSOM CANAL, CALIFORNIA.

PLATE XXXVIII.
TRANS AM SOC CIV ENGRS
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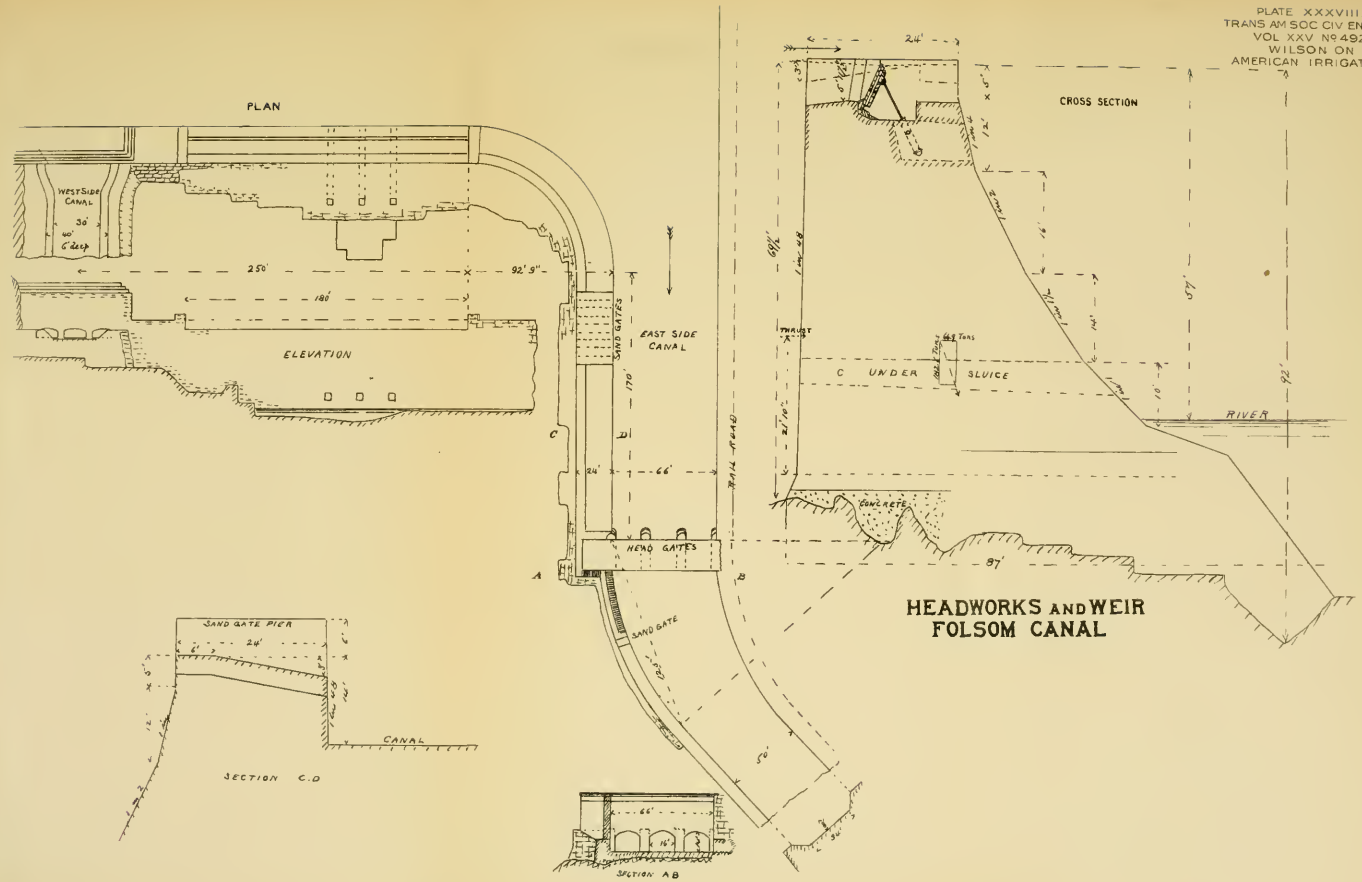
WILSON ON
AMERICAN IRRIGATION



it consists of a single Pratt iron truss the full length of the opening and of equal depth. This shutter is opened and closed by means of a row of hydraulic pistons which are operated from an accumulator placed near the power-house which is at the State Prison. It is thus possible, under pressure of the greatest head, to manipulate this shutter with ease. In like manner the gates closing the head regulator, the sand-gates and other similar works, are all operated by hydraulic power from the same accumulator.

Heading from both banks of the river, at either end of the weir, are a west and east side canal. The construction of that on the west side has not yet been commenced, and will not probably be undertaken for several years, until the irrigation resources on the other side have been more fully developed. Immediately in front of and above the head of the east side canal are a set of four under-sluice gates placed 6 feet below the grade of the canal and discharging back directly into the river. These gates, like the regulating head and all the first portion of the canal line, are constructed, as is the weir, of massive granite masonry. Each of these under-sluice gates is 5 x 6 feet in the clear, and opposite and below them is a sub-grade in the canal-bed across its entire width, in which sediment will be collected and from which it will be sluiced out by opening the gates. The head-gates themselves are three in number, each 16 feet in width and 12 feet high, and are of wood heavily trussed with iron and sliding between granite piers and abutments. Below the head-gates the first mile and a half of the canal, as far as the town of Folsom, is excavated on a steeply sloping granite hillside. This canal is 8 feet deep, 34 feet wide at the bottom, 50 feet wide at top, and is given a grade .0005 in 1, which produces a velocity of 3.6 feet per second and a discharge capacity of 1 210 second-feet. The banks of this canal and the dry rubble outer wall are given a slope of 1 on 1, the slope of the country rock above the maximum water line of the canal being $\frac{1}{2}$ on 1 and the top width of the lower retaining wall 10 feet. In the first 1 700 feet of the canal are seven separate sand-gates, each 5 feet wide by 10 feet high, and similar in construction to the scouring sluices and other works. These are of masonry set in the loose rubble bank on the lower side of the canal and discharge into the river. Their object is to clear the water as far as possible from sediment before it is passed through the turbines.

Combined Storage Works.

The storage of water for the purpose of supplementing the flow of intermittent streams, and thus increasing the amount of water available for irrigation during a certain short period of time, has been practiced as long in this country as has irrigation from perennial streams. The general principle of water storage for purposes of irrigation is so similar to that of storing it for domestic use that it seems to require little discussion in connection with an article of this kind. At the same time this subject has created so much discussion and comment in the last few years among legislators, capitalists, and engineers, that it is desirable to describe the methods employed and give details of some of the more important works completed or under construction in this country.

Since the passage by Congress of the Act of October 2d, 1888, providing for the survey by Government engineers and the withdrawal from occupation of lands included within storage reservoir sites, the growth of popular interest in the subject of water storage has received a marked impetus, and has led to the development of a great number of storage projects, a few of which are now under construction. Many of these are within themselves excellent and feasible projects and will some day be undertaken, while a large proportion will necessarily in the course of time prove impracticable and be abandoned. Connected with this bill were certain laws instructing the Director of the United States Geological Survey to outline irrigation projects and to designate for withdrawal from sale or occupancy such Government lands as might be reclaimed by future irrigation enterprises. For nearly two years active and important work was conducted to this end, both in the designation of irrigable land and reservoir sites, and in the survey and designing of many important canal and reservoir projects which will at some future day be constructed. The unfortunate clause directing the withdrawal of irrigable lands from sale or occupancy created the most violent opposition to this law, which, while it came from the west, was instigated by eastern capitalists interested in cattle raising or land speculation, who were thus deprived of the opportunity to acquire land with which to carry on their business. Much of the opposition to this law arose also from a misinterpretation of the effect which it would have on the acquirement of land by irrigation companies, and the perpetuation of this mistake by interested parties. Some of it came also from short-sighted merchants and real estate agents in western towns, who, while

they feared that the closing of agricultural lands to settlement for a short period of years would retard immigration and injure the present growth of the west, failed to appreciate the great benefits which must ultimately accrue from such legislation, in the more healthy and permanent growth which must result from intelligent irrigation development and legislation.

One good effect of this first law was, that though it was repealed, it has resulted in marked modifications and improvements of the land laws as they previously existed, reducing the amount of land which may be acquired by a single settler from 1 120 to 320 acres, and causing the repeal of the Pre-emption and Timber Culture Acts, and a marked reduction in the amount of land which may be procured under the Desert Act, as well as a decided improvement in the wording of that act itself. If the Government agitation on the irrigation subject had gone no further than this, it would have done a great good. It has, however, done much more, for it has resulted in the inauguration of a series of hydraulic investigations which have largely increased our knowledge of the evaporation, run-off from catchment basins, and the discharge of many of the larger streams of the arid region; and it has resulted above all in giving an added impetus to the construction of contour topographical maps in that region on a larger scale than that on which they were previously made, thus facilitating the discovery and projection of storage reservoir and canal enterprises, and adding largely to our knowledge of the feasibility of these, by showing the areas and slopes of the catchment basins of the different streams on which they are located. Above all, however, the retention of one of the most important provisions of the original bill, that which authorizes the survey and withdrawal from occupation of storage reservoir sites, lays the groundwork in the future for the development of many valuable water-storage projects; for it prevents agricultural settlers, and above all cattle and land speculators, from grabbing these sites, and then holding them back that they may charge *bona fide* irrigation companies such exorbitant prices for them as to either impede or prevent the development of their operations.

A notable instance of the damage done by such land speculators, is that of the parties who occupied some of the lands which would be flooded by the Sweetwater dam in San Diego County, Cal. These individuals kept very quiet regarding their operations and intentions during

the construction of that work, but on its completion they brought an injunction against the irrigation company to prevent their permitting the water to rise high enough to flood back on their lands, and asked such an exorbitant price per acre for what, until the construction of that reservoir, was barren arid land, as to make its purchase almost prohibitive. The case dragged for several years through the various courts, and each time a decision was rendered in favor of the landholders they raised the price of their property, in consequence of which the Sweetwater reservoir was never more than half full until within the last few months, when a compromise was effected between the contending parties.

In such an article as this, space, and the subject chosen, will not permit of a detailed discussion of the laws governing the discharge from catchment basins, the subject of evaporation, the considerations governing the location and the selection of reservoir and dam sites, nor the innumerable hydraulic problems connected with the designing of storage projects. Accordingly, as was done with the subject of canals, this portion of the paper will be confined to a description of the engineering details of various important storage projects completed or under construction for purposes of irrigation, taking these projects up progressively in the order of their importance and excellence of design, while dividing them among the two principal heads of earthen and masonry dams.

On the comparatively level bench-lands in Colorado, which slope away to the eastward from the foot of the Rocky Mountains, are a number of natural depressions which have been converted by the excavation of deep cuts or the construction of embankments into small storage reservoirs, into which are discharged the waters from the canal lines on which they may be located, and in which these waters are stored until wanted for irrigating the lands situated below them. Several reservoirs of this type have been designed and constructed in other parts of the west, notably, one on the line of the Florence Canal in the Gila Valley in southern Arizona, one on the line of the projected Sun River Canal in northern Montana, and a few others in other localities. It is not necessary to describe these works in detail, their construction is of the simplest, while the general plan on which they are projected is not the most commendable. It is not desirable to store water in the lower and more open plains country, nor to make use for purposes of storage of

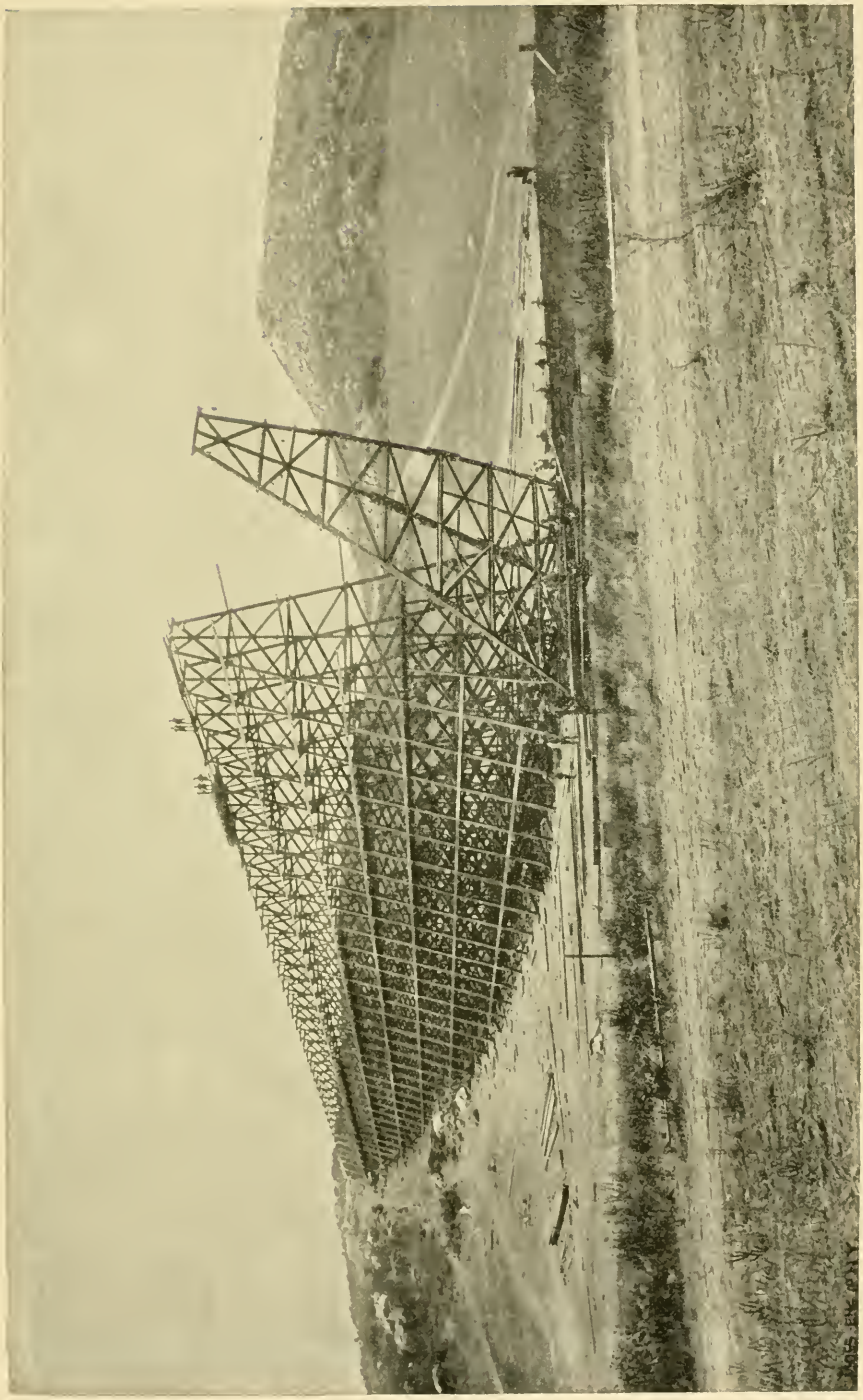
natural basins or depressions, because the loss by evaporation at the lower altitudes is great, and because the discharge capacity of the canal from which these basins are filled must be made considerably greater than would be necessary if they were merely used as direct irrigation canals. Storage reservoirs of the other class, those which are located at higher altitudes and among the more rugged mountainous regions, have several advantages. The loss by evaporation from their surfaces is reduced to a minimum. The water which is stored in them is comparatively free of sediment and their life is correspondingly increased. All the water which runs off their catchment basins flows immediately into these reservoirs without the interposition of any artificial channel, and it can accordingly all be stored, providing the capacity of the reservoir is sufficiently great. And above all, sites for these reservoirs can usually be found where, by the construction of short and high dams the water surface exposed both to evaporation and absorption is reduced to a minimum.

One of the first reservoirs of this type constructed in this country, which furnishes a good type of earthen dam and combined natural and artificial distributing system, is the Cuyamaca reservoir, situated in the Coast Range, about 70 miles east of San Diego, Cal., at an altitude of 5 500 feet. This work is the property of the San Diego Flume Company which was organized in 1886, with the object of bringing water from the high Coast Range mountains for the supply of San Diego City with domestic water, and to furnish the ranches in its neighborhood with a sufficient water supply for purposes of irrigation. The system consists of a storage reservoir having a capacity of 830 acre-feet, from which water is turned back into Bowlder Creek, down which it flows for a distance of $12\frac{1}{4}$ miles with a fall of 4 000 feet to the main diverting dam, the altitude of which is 800 feet, and above which the diversion canal heads. Owing to the value and scarcity of water in this neighborhood the canal is not in open excavation, as the loss by evaporation and absorption in such a channel would be too great, but it consists of a wooden flume 36 miles in length, built for a majority of that distance along the steeply sloping cañon sides of the San Diego River, and requiring a number of trestles and tunnels in its course in order to cross side-drainage channels, or to pierce long bends which would have greatly increased its length. In all there are 315 trestles, having a total length of 38 042 feet and heights ranging from a few feet to 85 feet, the longest being that across Los

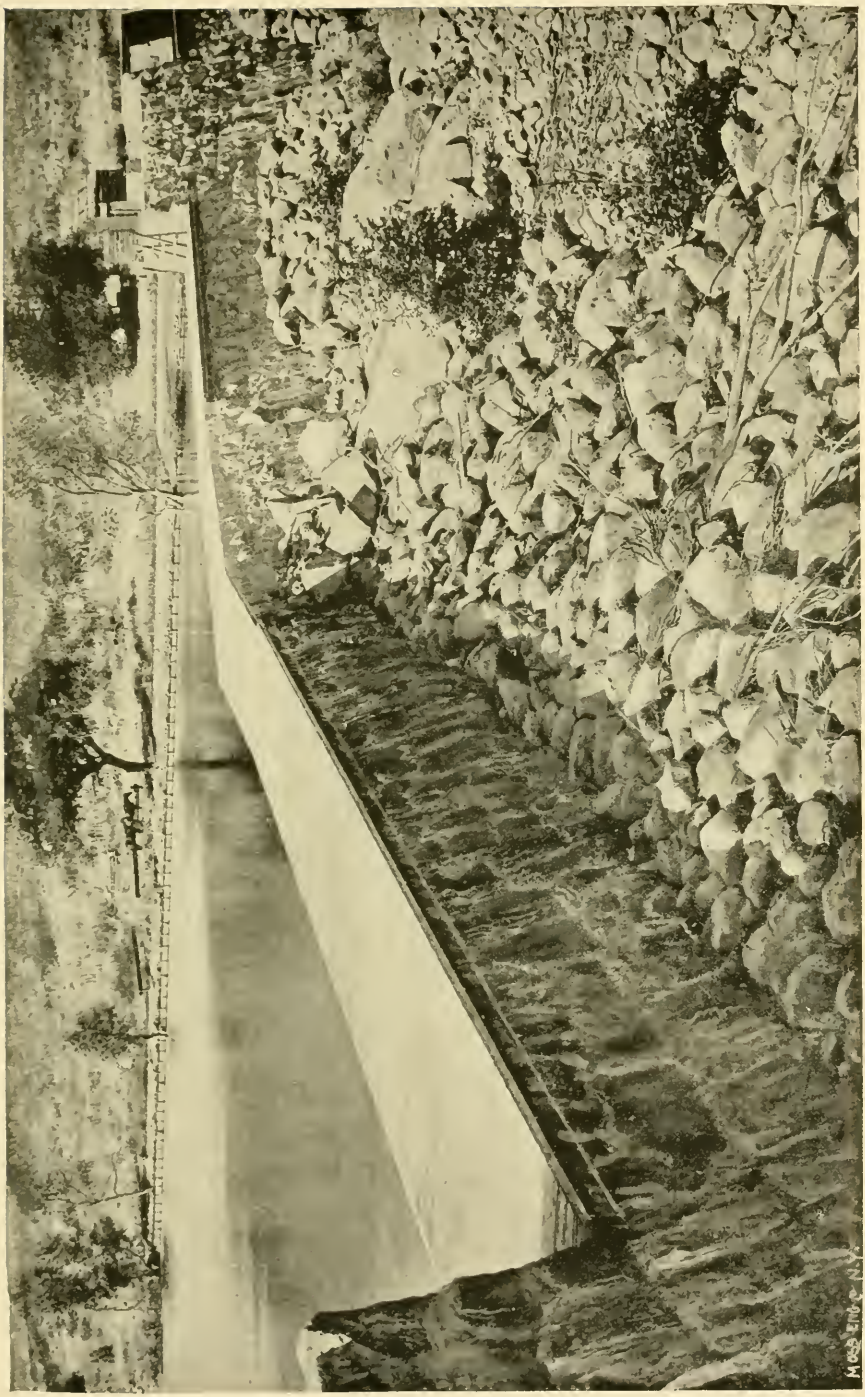
Coches Creek 1 794 feet in length (Plate XXXIX) shows the work during construction. There are also eight tunnels pierced in rock or earth with lengths varying from 9 to 1 901 feet, the total length of which is 4 190 feet, with 4 760 additional feet of masonry approaches.

The Cuyamaca Reservoir, situated on the head-waters of Bowlder Creek, a branch of the San Diego River, has a catchment basin of 11 square miles on which the annual average precipitation is about 45 inches. This reservoir is formed by a dam 700 feet long and 35 feet wide on the crest; 140 feet wide at its base and 35 feet in maximum height. The water slope is 1 on 2 and the outer slope 1 on 1½, while the inner slope is covered with a rip-rapping of stone 2 feet in thickness. This dam is composed of a hard clay which was accidentally found below the surface while excavation was in progress for the foundation. It rests on this foundation bed of clay which is over 12 feet in depth, into which a trench 8 feet in depth is excavated and filled with a puddle of one part of small broken stone, one part sand and three parts clay, the whole well rammed. This puddle wall is carried up through the entire height of the dam, having a top width of 6 feet and a batter on both faces of 4 on 1. The surface of the ground under the inner slope of the dam is covered with 2 feet in depth of clay puddle well bonded. The chief element of weakness in this dam is the discharge sluice, consisting of a trench filled with puddle, on which 18 inches of concrete is laid, and above which the masonry culvert is built up. Owing, however, to bad bonding with the surrounding material of the dam, a considerable amount of leakage occurred at first, following the line of this culvert. This has been recently diminished, however, by the addition of ribs to the outer circumference of the culvert and the adoption of other remedial measures. A little beyond the south end of the dam a waste-way is constructed 50 feet in width and 4 feet deep, through which the velocity of the water is about 5 feet per second. This waste-way is ample to discharge the greatest flood which may fall upon the catchment basin of the reservoir.

After flowing down Bowlder Creek and the San Diego River for 12½ miles the water is diverted into the San Diego Flume by means of a great pick-up weir built of uncoursed rubble masonry, which extends across the entire width of the San Diego River. This weir shown in Plate XL is built in two tangents, the exterior angle of which points up-stream. Near its south end and extending out

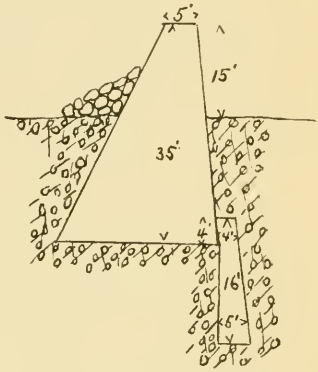


ERECTION OF HIGH TRESTLE ON SAN DIEGO FLUME, CALIFORNIA.



DIVERSION WEIR, SAN DIEGO FLUME.

for a distance of 108 feet to the head or regulating gates which admit the water to the flume, the cross-section of this weir is 4 feet wide on top, or 1 foot less than that of the remainder of the weir. At a distance of 32 feet beyond the flume head is an open waste-way 20 feet wide, the crest of which is 4 feet lower than that of the remainder of the weir. For 14 feet beyond this waste-way the weir is again given its usual height as far as another waste-way 165 feet in length, the crest of which is, like that of the first described, 4 feet lower than that of the remainder of the weir; the remaining 15 feet in length of the weir is again full height. Between the two waste-ways just described and in the bottom of that portion of the weir which is built to the maximum height, also, under the regulating gates at the head of the flume, are two single under-sluices opened by means of gates which slide vertically and are operated by means of a screw and hand-lever from above. One of these under-sluices has its sill 14 feet and the other 18 feet below the crest of the dam. The cross-section of the weir, a diagramatic sketch of which is shown, is peculiar, and is the result to a certain extent of a failure on the part of the builders to found it on bed-rock. The first wall as originally built was 35 feet in height, 5 feet wide on the crest and 16 feet wide on the base, and was sunk to a depth of from 15 to 25 feet into the gravel of the river bed, presumably to bed-rock. It was soon discovered, however, that a considerable leakage passed under this weir, and after examination by competent engineers it was decided to build a sub-wall on the up-stream side across the deepest part of the river channel as indicated. This wall is 16 feet in height, but it is claimed by those who were present during its construction that it was never carried down all the way to bed-rock. This belief is further exemplified by the fact that there appears to be considerable leakage under the weir, as the discharge of the river below it is claimed to be greater than it was before the weir was constructed.



The regulator at the head of the flume consists of two gates, the sills of which are about 10 feet below the crest of the weir. These are 40 inches each in the clear, and are raised by means of a screw and hand

lever operated from above. The flume which transports the water to the irrigable lands in the neighborhood of San Diego is well constructed, is entirely of wood, and is everywhere laid in excavation or on trestles, in no place resting on embankments. The excavation in which it is laid is 12 feet wide, or 6 feet wider than the flume, thus giving a place in which loose rocks and stones sliding from the hilltops above can lodge without injury to the structure. The flume is 6 feet wide in the clear with sides now 16 inches high, though the side posts and framing are 4 feet in height, so that whenever the demand for the water requires it, two further rows of planking will increase the capacity of the flume by that additional depth. The flume rests on transverse sills of 2-inch planks laid 4 feet apart, and on these rest 4 x 6 longitudinal stringers, above which is constructed the frame work of the flume with its lining of 2-inch planking. At a point about 200 yards below the head-gate is constructed an escape or waste-gate, 4 feet in width, which discharges into a short flume 50 feet in length, by which the waste water is led back to the San Diego River. The grade of the flume is 4.75 feet per mile, its maximum capacity is estimated to be 50 second-feet, and its total length is 34.45 miles.

Another and the most modern storage reservoir constructed in the west, the dam of which is composed of earth, was built by the Crocker-Huffman Land and Water Company, about 5 miles northeast of Merced, Cal. This project consists of a temporary diversion weir on the Merced River, about 25 miles above the city of Merced; of a canal 27 miles in length, which leads the water into the reservoir; of the storage reservoir itself, and of the distributing canals leading from it to the irrigable lands. The Merced reservoir, the surface area of which is 500 acres, is closed by a dam 4 000 feet in length, and 54 feet in maximum height, the top width of which is 20 feet, and the capacity 15 000 acre-feet. For 2 600 feet of its length the dam is quite high, the remaining 1 400 feet being a mere embankment not over 10 feet in height. Its inside slope is 3 to 1 and the outside slope 2 to 1, while the inner slope is rip-rapped for 12 inches in depth with stones roughly laid on for a depth of 15 feet below the crest. The catchment area of the reservoir proper is so small that it was not considered necessary to provide any other escape than the outlet tunnel of the reservoir, which has a capacity of about 100 second-feet. A mile above the reservoir on the main canal line is an escape discharg-

ing into a natural channel, and at an elevation of 1 foot above the canal sill at the point where it discharges into the lake, so that if the lake were exceptionally high, this escape would be brought into operation. According to Mr. C. D. Martin, the engineer who constructed this work, the dam is composed of a sandy clay laid dry in layers, and well tramped over with scrapers and horses. The foundation is of the same material, and test borings showed its depth to be at least 15 feet. The discharge pipe is 2 feet in diameter, and is constructed of $\frac{1}{2}$-inch cast-iron laid in a brick-in-cement tunnel 4 feet in diameter, with walls 1 foot in thickness. Every 15 feet on the outer circumference of the tunnel are projecting rings of masonry 1 foot in thickness and 2 feet in height, the object of which is to prevent the travel of seepage water. In addition to the outlet pipe which carries water under pressure to the city and to some of the surrounding irrigable properties, there is at the south end of the dam an ordinary discharge gate, set a few feet below the maximum surface level of the reservoir, which leads into a distributing canal commanding a large portion of the irrigable lands.

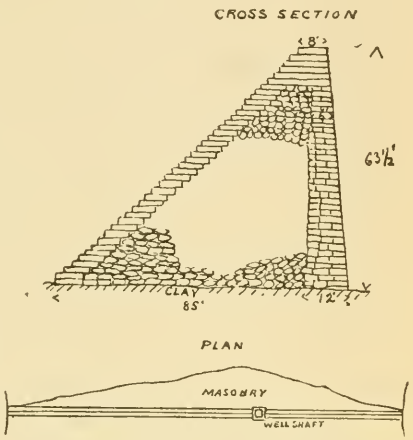
The headworks of the feeder canal are of the simplest and crudest description, consisting of an old weir built of brush, rocks and crib-work, the total length of which is 250 feet, and the height 12 feet. The canal heads 200 feet above the dam at a slight angle pointing upstream, and its entrance is closed by a set of sixteen regulating gates, each 5 feet wide in the clear and 20 feet in height above the canal bed. This regulator, which is formed somewhat like a long flume with gates in it, has a floor of 3-inch planking extending 20 feet upstream and 10 feet down-stream. Anchor and sheet piling is let 4 feet into the rock and cemented under the upper and lower ends of the floor and under the gates, the latter being simple wooden structures which are lifted by means of a hand-lever operated from above. The first main portion of the canal, which is 7 miles in length, has a capacity of 1 500 second-feet and a bed width of 60 feet, with slopes of 2 to 1 inside, and $1\frac{1}{2}$ to 1 outside, a depth of water of 8 feet and a grade of 1 foot per mile. This portion of the canal is excavated mostly in sand and gravel, while the remainder of the canal is generally in a fairly good soil. On the line of the canal are two tunnels, the first 1 700 and the second 2 000 feet in length, both are 20 feet wide at the bottom, 14 feet in height at the summit of the arch, with a grade of $10\frac{1}{2}$ feet per mile. The first tunnel is excavated entirely in sandstone and requires no lining, while

the second is in friable sandstone or in gravel, and is timbered throughout. Below the first tunnel is taken off the first main distributary which discharges into a creek, the line of which is utilized as a canal. The practice of utilizing stream beds as distributary channels is employed generally on this canal, and is not one that is to be commended. Experience has shown both in this country and abroad, that the loss by evaporation and absorption is greater in a natural channel, the grade and bed width of which is not exactly that which theory would require, while the difficulty and expense of diverting water from such stream beds to the private irrigating channels form an additional objection to their use. Moreover, the employment of the natural drainage lines of the country as irrigating canals impedes the proper circulation of the irrigation waters and gives rise to the production of alkali and malaria.

A very interesting composite irrigation scheme, combining, as it does, two different varieties of water storage, is the project of the Denver Water Storage Company, the main reservoir for which has just been completed. This company acts in co-operation with two other corporations, one of which, the Denver-Arapahoe Land Company, controls the lands which this project will irrigate. The system comprises a main storage reservoir, known as Castlewood Reservoir, situated on Cherry Creek, at a narrow point in the cañon and about 30 miles southeast of the City of Denver; of a pick-up weir a mile and a half lower down on Cherry Creek which will divert the water which is turned into Cherry Creek, and pass it into the Arapahoe Canal which heads at that point; of the main line of the Arapahoe Canal which conveys the water to the irrigable tract situated immediately southeast of Denver and West of Cherry Creek; and of a series of four secondary reservoirs situated in different parts of the irrigable lands which are formed by natural depressions, and one of which has been completed and is now being utilized. This system of secondary reservoirs will be filled by water brought down by the canal at such times as it is not wanted for the irrigation of the land, or when there may be a superabundant flow in Cherry Creek which would otherwise be lost. The area of good irrigable land which is commanded by this system as at present constructed is 30 000 acres, and the approximate cost of the canal and main reservoir is about $305 000.

The capacity of the reservoir is 5 300 acre-feet, and its catchment area is 200 square miles, the available run-off of which is estimated to be

about 2 300 acre-feet per annum, while the maximum flood anticipated is 7 000 second-feet. The construction of the dam is peculiar (see sketch) and has been severely criticised by the people living below it and by the inhabitants of Denver as unsafe, and injunctions have been taken out to prevent its being filled with water. It is founded on a bed of clay and bowlders from 7 to 30 feet in depth, and is composed of an outer shell of large coursed blocks of rubble masonry, the thickness of which on the up-stream face is about 6 feet on top and 12 feet at the bottom. On the down-stream face it is from 5 to 7 feet in thickness, this facing being laid in steps as shown in the accompanying diagram, the height of the steps varying from 1½ to 2½ feet. The main body or center of the dam consists of dry-laid rubble and stone, inclosed between these two masonry shells. The maximum height of the dam is 63½ feet, its crest is 586 feet long, and 100 feet of this length is lowered 4 feet in order to form a waste-way over which flood waters may be discharged. In addition to this waste-way is another around one end of the dam, which is 10 feet wide and has a fall of 4 feet in its length of 400 feet discharging back into Cherry Creek below the dam. The upper 4 feet of the dam is vertical on both sides, 8 feet in width, and constructed of rubble masonry, the outer slope of the remainder of the weir is 1 on 1, while its inner slopes is 10 on 1. The well-shaft for controlling the outlet sluice is constructed of the best masonry and is built into the dam close to its upper slope, the total thickness of the masonry wall on both sides of this shaft being 4 feet. Discharging into the shaft are four sets of iron intake pipes set in cement, the lower set being on a level with the bottom of the reservoir and the others each 6 feet apart vertically. These discharge into a central well from which an outlet pipe 3 feet in diameter leads the water off, the pipe being surrounded by 4½ feet in thickness of concrete.



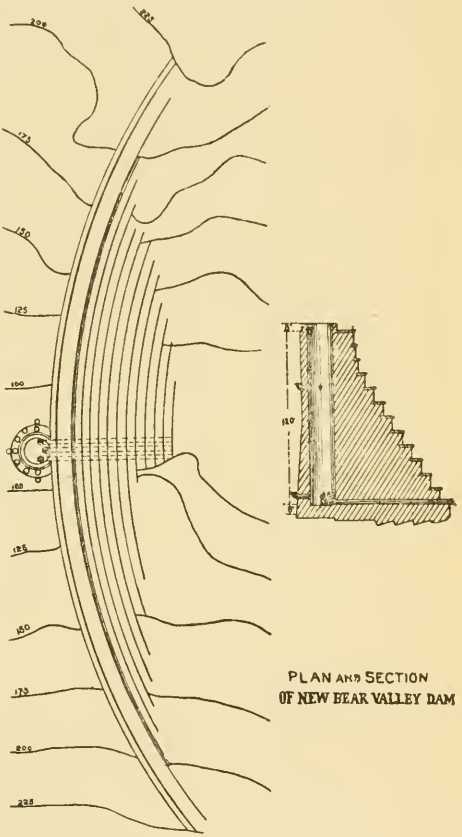
After being discharged into Cherry Creek the stored water runs down

the bed of that stream for a distance of 1½ miles to the pick-up weir, which diverts it from the left bank into the canal. This weir is 150 feet long, 10 feet high, and, with the exception of 54 feet of its length in which the head-gates are situated, it is composed of piling sheeted on both faces and forming a rectangular wall which is filled with sand. These piles are 26 feet long, rest on bed-rock, and are placed 6 feet apart across-stream, and 3 feet between centers in the direction of the stream. Through the center of this portion of the weir is a waste-way 60 feet in width, with its crest 3 feet lower than that of the remainder of the weir. The head-gates, as before stated, are taken out of that portion of the weir which is constructed of rubble masonry, and are three in number, their total height being 6 feet, and their width between centers 4 feet. The masonry section of the weir in which these gates are located is 3 feet wide on top, its inner slope being 1½ on 1, and the outer slope vertical.

The canal by which the water is led into the secondary reservoirs and to the irrigable lands, has a total length of about 50 miles including its main branches; the first 25 miles of this line is 12 feet wide on the bed, has 3 feet depth of water, slopes of 1 on 1½ and a grade of 1½ feet per mile, giving it a discharging capacity of 75 second-feet. In the next 10 miles the canal has a bed width of 10 feet and the remainder of its course has a bed width of 8 feet, the slopes and grade remaining the same. The majority of the line of this canal is constructed in an easily worked clayey loam.

The secondary reservoir which is already constructed has a surface area of 60 acres and a capacity of 700 acre-feet, and as before stated, consists of a natural depression the maximum depth of which is 16 feet. This reservoir is emptied by means of a wooden outlet pipe let into the embankment which closes the lower side of the basin. This earthen embankment is 10 feet in maximum height, 12 feet wide on top and has slopes on the inside of 1 on 3 and outside of 1 on 1½. On the line of the feeder canal are twenty flumes crossing smaller side drainage lines, the greatest being 200 feet in length and the greatest height 12 feet. Nearly all of these flumes are furnished with escape gates by which surplus water can be discharged into the creek. In addition to these there are 5 300 feet of inverted wooden syphon pipe line for crossing ditches or drainage, the longest syphon being 1 800 feet in length with a maximum depth of 100 feet, giving a water pressure of 50 pounds per square inch.

The Bear Valley Dam, which is situated in the San Bernardino Mountains east of San Bernardino, Cal., has been so frequently described because of the peculiarly bold cross-section given to it, that it will be unnecessary to refer here to any of the details of its construction. In connection, however, with the company which built this work, there have recently been inaugurated a series of large irrigation projects which will necessitate the construction of a greater storage reservoir than that now existing. The company which is now developing this property is known as the Bear Valley and Alessandro Development Company and is under the general management of Mr. Frank E. Brown, the builder of the original Bear Valley Dam, who also acts as consulting engineer for the new work. This company owns a large tract of land about 10 miles south of the City of Redlands, from which it is separated by a low range of hills. The water supply, however, must be derived from the Santa Anna River and its tributaries which flow through Redlands and on the headwaters of which is located the storage reservoir. The perennial discharge of this river is comparatively small, and is already entirely appropriated by companies having prior rights and operating in Redlands, Riverside and other localities; it accordingly became necessary to obtain storage water for the irrigation of the Alessandro tract.



PLAN AND SECTION OF NEW BEAR VALLEY DAM

The capacity of the present Bear Valley Reservoir is said to be 36 000 acre-feet. It is situated in a broad and flat mountain basin known

as Bear Valley, at an altitude of about 6 200 feet above the sea, and discharges into Bear Creek, a fork of Santa Anna River, through a very narrow granite gorge in which the dam is built. This dam (see sketch), which is founded on granite and abuts against the firm granite walls of the cañon, is 300 feet in length on the crest, and is curved with the convex side up-stream in the form of an arch, the radius of which is 335 feet. The cross-section of this dam is remarkable; its maximum height is 64 feet, the top width 3 feet and the lower face vertical for 48 feet, the upper face being battered so that at a depth of 48 feet it is 8.5 feet in thickness. At this plane there is an offset up and down stream giving the crest of this lower wall a thickness of 12½ feet. Below this plane both faces have a slight batter to the foundation, which has a thickness of 20 feet. The structure is constructed of rough ashlar masonry on both faces, filled with coursed rubble masonry in the interior, all laid in uniform beds of Portland cement. These stones are of granite and vary from 2 to 5 feet in length and about 1 to 2 feet in width.

The south end of the dam abuts against a massive ledge of granite standing out about 100 feet into the cañon. This ledge really forms a part of the dam and over it an escape-way has been cut having a width of 20 feet, with its sill 8.5 feet below the level of the crest of the dam. Through the bed rock, about 9.5 feet below the base of the dam, and one-third of its length from the southern end, is a cutting which forms a masonry-lined culvert 2 x 3 feet in dimensions which discharges into a masonry pool from which it is expected that the water can be measured over a weir. On the upper side this culvert narrows to its entrance which is closed by an iron gate 10 x 24 inches, which slides on brass bearings and is operated from above by means of a screw and wheel. Frail as is the cross-section of this structure, it is a remarkable fact that owing to the excellent manner in which it was built, and to the perfection with which it acts as an arch, it has stood now for several years, through as many years of heavy floods and shows no sign of weakness. The waste-way is ample for the discharge of all the flood waters, though enormous floods occasionally occur at the high altitude at which it is located. During the twenty-four hours included from 6 P.M. on February the 26th, to 6 P.M. on February 27th, 1891, a rainfall of 17 inches was recorded on the rain-gauge at the dam, while the total downfall which occurred during that week was, perhaps, as much as 40 inches.

The new dam, the money for the construction of which has already

been subscribed, will be a much larger and more substantial structure than the present one, and will be situated 150 feet below it in what is considered a more favorable location. Like the present dam it is to be constructed of the best granite masonry, and will be arched with its convex side pointing up stream. This dam is to be 120 feet in maximum height, the foundation being from 8 to 12 feet deeper and the coping 5 feet in height on the top of the crest. Its upper surface will have a very slight batter, and the lower surface, while its general slope is about the same as that which would be given by Molesworth's, Bouvier's, or some other of the modern formulas, will be built in steps having a uniform rise of 10 feet and an offset in proportion to the batter at that point. The top width of the dam will be 15 feet, its width at base 73½ feet, and the maximum pressure when full is estimated to be 11.6 tons per square foot on the toe of the dam, and when empty 9.3 tons per square foot at the upper side. The discharge sluices, which are three in number, head in a cylindrical gate-well which extends the entire height of the dam on its upper face. This gate-well will be 18 feet in diameter inside, and will have three inlet pipes at the bottom, four more 40 feet higher, and three more at a height of 80 feet from the bottom. The outlet sluices will consist of three cast-iron pipes, each 36 inches in diameter, the entrances to which will be closed by gates worked from above, and the total discharge of which is estimated to be 1 500 second-feet.

This great dam will flood the water back from Bear Valley proper over a low divide, into a smaller basin which is situated on the eastern watershed of the San Bernardino Mountains. The area which will be flooded will be 7 850 acres, and the maximum capacity of the reservoir is estimated to be 322 000 acre-feet. The waste-weir for the discharge of surplus water in time of flood will be located at the opposite end of the valley from the dam at the lower end of the second basin, and will discharge toward the eastern plains. The capacity of this reservoir is so great that the watershed of Bear River, which is only 77 square miles, is insufficient to fill it; accordingly a canal will be constructed along the eastern slope of the mountains which shall catch the water of White River, Mission Creek and several minor streams, and lead it into the reservoir at its eastern end. This feeder canal will have an ample cross-section to carry the ordinary flood water of these streams and will be about 18 miles in length.

When wanted for purposes of irrigation the water stored in this re-

servoir will be discharged into Bear Creek, down which it will flow a distance of 12 miles to the Santa Anna River, and an additional 15 miles down this stream to its exit from the foot-hills, at which point it will be diverted. In its course down these streams the water flows over a very steep bowlder-bed with a total fall of about 5 200 feet. At present there are diverted from the Santa Anna River two small canals, one from either bank, which lead the water to the irrigable lands in the neighborhood of Redlands. The new project will carry the water, by means of a pipe-line 10 miles long, to the Alessandro tract.

This pipe-line is now in process of construction and is nearly completed. The diverting dam at the head of the pipe-line has not yet been constructed but it will be a substantial and well designed structure, and immediately above it the pipe-line heads, having a perfectly straight line for a distance of about 2¼ miles to the crossing of Mill Creek, in which distance it passes over some hilly country. From there on it follows somewhat the contour of the country to the divide before spoken of as separating the Santa Anna Valley from the Alessandro tract. This pipe-line is of steel, 28 inches in diameter at first and afterwards reduced to 24 inches. The tunnel through the divide is 2 300 feet in length. It is cut through hard rock, the floor and sides of which will be lined with a coating of asphaltum. Beyond the tunnel will be 2 600 feet of open canal, likewise asphaltum lined, after which will be laid about 150 miles of distributing pipe, some 14 inches in diameter, and the remaining branches about 9 inches in diameter. It is intended to pipe this water to the highest point of every 10-acre lot, whence it will be distributed to the plants by means of sub-irrigation through pipes, as is done in many other parts of Southern California.

It is not necessary in this article to more than revert to the Sweetwater Reservoir, which is situated a few miles southeast of San Diego, California, on the Sweetwater River. The magnificent dam by which this reservoir is formed has been ably described by Mr. James D. Schuyler, the engineer who constructed it, in a paper read before this Society in November, 1888.

The latest and most modern reservoir closed by a masonry dam which has been designed in this country is that on the Sharon estate in California, known as the Berenda-Chowchilla project, of which Mr. William

Ham. Hall is chief engineer. This tract consists of 30 000 acres for which a storage supply must be provided, as the perennial flow of the Chowchilla River is not sufficient to irrigate it. The project consists of a storage reservoir situated on Chowchilla Creek, from which the water will be discharged into that stream, and from which it will be diverted after flowing down it for 2¼ miles by a diversion weir; and of the main diverting canal which will lead the water to the irrigable lands. Though the perennial discharge of the Chowchilla River is small it discharges in times of flood as much as 1 500 second-feet, an ample volume to fill the reservoir within a few weeks.

The dam which is to be constructed of uncoursed rubble masonry is at an altitude of 420 feet above the sea and will form a reservoir having a superficial area of 100 acres and a capacity of 42 400 acre-feet, which it is believed will be sufficient to irrigate 26 000 acres of the tract. The maximum height of the dam is 100 feet, its length on crest is 780 feet, and it is curved up-stream with a maximum radius at the center of 1 146 feet, which radius is diminished gradually to 736 feet at the abutments, thus broadening the base on which it abuts against the hillsides, and thus increasing its power to act as an arch. Two waste-ways will be excavated, one at either end of the dam, the combined discharge capacity of which will be 15 000 second feet. The entire structure is founded on a firm dioritic rock which outcrops near the surface, and while its cross-section is very similar to that given by Wegmann's or other modern formula, it is made a little heavier than these would require, owing particularly to the increased width near the abutments. It has a uniform batter on the up-stream face of 1 in 25 below the first 10 feet, which are vertical, its down-stream face being curved according to theory and producing a maximum width at its base of 68 feet. The top width of the dam is 10 feet, which will be widened by a cornice and a 4-foot parapet to 13½ feet. There will be two outlets or discharge channels excavated in solid rock under the dam at either side of the stream-bed, and each 20 feet above its base. These discharge sluices will each contain a steel pipe laid in cement and operated by valves from above. In addition to these there will be an under-sluice 6 feet above the bottom of the dam, also laid 10 feet deep in the rock under the structure and in which will likewise be laid a steel pipe.

Water will be diverted from Chowchilla Creek by means of an uncoursed rubble masonry weir 12 feet in maximum height and 400

feet long on the crest; this will be a clear overfall weir, and is located on a reef of rock in which is a natural depression at the south end of the weir, which will be utilized as the canal-head. This depression is 30 feet in width below the crest of the weir and the main canal will head at this point. This canal will have a capacity of 300 second-feet and will be 4½ miles in length with no difficulties of construction on its line. At the end of this distance it will be divided into several distributing branches which will convey the water to the irrigable lands.

In addition to the works described in this article there are numerous other important projects either completed or under construction which illustrate the magnitude and importance of the present stage of irrigation developed in our West. Some of these works are quite as large and as well worthy of note as those which have been described, though few of them contain as many important and typical engineering works as those mentioned. Among the more important canals may be mentioned the Eddy Canal, which is diverted from the Pecos River in New Mexico, at the head of which is a great diverting dam constructed of loose rock with an earth backing, the maximum height of which is 54 feet and the length on the crest of which is 1 100 feet. This dam also creates a storage reservoir which supplements the discharge of the river. The canal itself is 50 feet wide and 6 feet deep, about 30 miles long, and on its line are some very heavy rock works and a high terre-plein and aqueduct crossing the Pecos River.

Another important canal, which is also nearly completed, is diverted from the Sacramento River above the Town of Colusa in California, for the irrigation of the Central District. This canal has a capacity of 750 second-feet; is 60 feet wide at the bottom; carries 6 feet in depth of water, and will command about 156 000 acres. The first 6 miles of its length are in earth excavation, the maximum depth of which is 19 feet, and at its head and on its line are several important and substantial structures, such as regulating gates, falls, etc.

Among the other important reservoir projects of which no mention has been made, is that of the Citizens' Water Company, near Denver, Colo., the primary object of which is for the water supply of the City of Denver. This reservoir will be closed by a mammoth earth dam 261 feet in height and 30 feet in width on top; the side slopes of which are 2½ to 1 inside and 1½ to 1 outside, set back with several benches or bermes, each 10 feet wide, to prevent slips.

DISCUSSION.

Colonel WILLIAM E. MERRILL, M. Am. Soc. C. E.—Is there any difference in the effect of irrigating with clear or with muddy water?

Mr. WILSON.—There is a considerable difference of opinion on that subject. The Indian practice is to try to get the clearest water they can. They use their main canals very much as reservoirs and take their drainage canals off at some feet above the level of the main canal, as high as they can, in order to get the clearest water possible. In our Western States the practice is exactly the reverse of this; they do not object to having sediment in the canals; they think it helps the canals, providing they can keep the sediment in suspension, and allow it to be deposited on the lands. In the case of the Turlock canals, the main canal, its branches and ordinary distributaries are so arranged, that the sediment will be held in suspension until it reaches the irrigable lands, as this is considered desirable.

A MEMBER.—What is the usual method of utilizing water from the canals? Do they overflow the land with the water, or simply dig a few ditches and let it go through the soil?

Mr. WILSON.—I have not touched upon that point because I considered it was not connected with irrigation engineering. As the article is rather long, I have avoided touching upon a good many features of irrigation.

The practice differs considerably in different parts of our country, but the water is utilized, I may say, by three methods. In a gently sloping country, like California, the check system is most employed. This consists of building up a series of embankments about 1 or 2 feet in height, by means of scrapers and plows. The water is poured from the ditches above each successive check, by which it is held up and is allowed to soak into the soil. In most parts of the country the general practice is to construct little branch canals to the various plots to be irrigated, and if the crop is wheat or grain or grass, they so plow their fields as to leave ditches about 10 or 12 feet apart, into which water is turned. These are laid out on a proper grade. The water goes into them and is absorbed by the soil; then they break away the next check, and allow it to flow down to the next ditch. The second method is to flow the water over the land and to a depth of 6 inches, and allow it to lie and to soak into the ground. The practice is not to irrigate more than three to five times a season.

In Southern California subirrigation is now rather extensively practiced, by a system of pipe lines that lead to every individual tree and have an outlet under the tree, and the water is absorbed in the soil by capillary attraction; in that way the duty for water is the highest

that is obtained anywhere. In the case of the rice crops in India the land is flooded to several feet in depth.

JAMES B. FRANCIS, Past-President Am. Soc. C. E.—What means do they take in the earthen dams to secure tightness? By means of core walls?

Mr. WILSON.—The practice in the western States is similar to that of Europe and India. I know of no earth dam in the west which has either a masonry or a wooden core. I do not know of any work which is considered a good work which has a puddle wall in it. Sometimes they excavate puddle trenches, down to some impervious strata of clay, and the puddle wall is carried up a few feet into the embankment, but the embankment itself is usually built in layers and consists of a clayey soil, wherever possible, with gravel and small stones mixed in. In the case of the Cuyamaca dam there is a certain proportion of clay—apparently mixed with gravel—and small stones, properly mixed. In the case of the Modesta dam, where floods are not feared, the common soil was tramped over and worked down in layers.

A MEMBER.—I understand they do not roll the layers.

Mr. WILSON.—They did not.

WILLIAM P. HARRIS, M. Am. Soc. C. E.—I understand that the slopes are 1½ on one side and 2½ on the other.

Mr. WILSON.—Not on the reservoirs; on the canals the slopes are steep. On the reservoirs mostly 2½ and 3.

Mr. HARRIS.—That seems to me astonishing. The levees in the Mississippi River are built entirely now of slopes of 1 on 3, and engineers prefer to build them 1 on 4. They are pretty well trodden down and tamped, but they are given a slope of 1 on 3, sometimes 1 on 4 and even 6; in case of extreme height they have a bank on the land side of about 20 feet, but no such slope as 1 on 1½. The material varies from a pretty clear sand to a pretty clear clay.

G. BOUSCAREN, M. Am. Soc. C. E.—What kind of timber do they use in a flume canal and how long does it last?

Mr. WILSON.—It differs considerably with the locality. In California, take the San Diego Flume Company's work—the trestle and the sills and all the timbers on that work are of Oregon pine, and the planking is of redwood. Redwood is supposed to last for a very long time. The San Diego flume has been constructed about eight years and it does not show the slightest sign of deterioration so far as I was able to discover. It is laid everywhere in excavation on a narrow embankment, and there is about 4 feet width of excavated berme on which it rests on each side of the flume. The bench flume in the Highline Canal in Colorado has been in operation about ten years. It is built, I think, of Colorado pine, and that flume shows no great deterioration of the trestles and other timbers. A good deal of the planking has been replaced, but I suppose

a very small percentage of the entire flume has been repaired. I should judge their life would average ten to fifteen years.

JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—Mr. Wilson mentioned a great difference in the expansion and contraction of the aqueduct; how was that?

Mr. WILSON.—They have overcome it, as I recollect—(I do not wish to be put down as saying it is so)—largely by building the wooden flume well into the iron aqueduct and by making leaden connections. The engineer in charge criticised the iron aqueduct, and said it ought to be replaced by a wooden one. The great aqueduct is simply an iron trestle on which rests a wooden flume, so that the water channel is a wooden flume throughout, both on land and on trestle.

Colonel MERRILL.—Judging from the experience of others I should conclude that there was no insuperable difficulty in the use of iron aqueducts. I have in my office a photograph of an iron aqueduct on a navigable feeder of the Central Canal of France, and from what I know of the minute attention given to details by French engineers, I should conclude that satisfactory means had been found for making the ends water-tight. The clear width is 10 feet and the depth of water is 5 feet. At the famous hydraulic lift at Fontinettes (also in France) the upper approach consists of two independent metallic aqueducts, with a span of 68 feet, a depth of water 6½ feet, and a clear width of a little over 17 feet. When I visited this work in 1889, I neither saw nor heard of any leak. In our own country there is an aqueduct on the Mussel Shoals Canal, where it crosses Shoal Creek, that is 900 feet long, 60 feet wide in the clear, and carries 6½ feet of water. I have never heard of any difficulty in keeping the ends water-tight.

Mr. WILSON.—On the Agra Canal in India, is an iron aqueduct, 10 feet in height and 20 feet in width, which is connected with the river and masonry abutments by means of sheet lead. There is no leakage of any consequence on this aqueduct.

Mr. FRANCIS.—I have had occasion to make large lines of pipe, from 3 to 6 feet in diameter, of boiler iron; these are connected with masonry at one end. Expansion would be a great detriment, so we provided for it by riveting to each of the pipe ends to be joined a plate with a wide flange turned outward at right angles. These flanges were riveted together near their edges, thus allowing for a certain amount of change of length.

Mr. TRAUTWINE.—Is not that the same form of joint that is used in the larger water pipes in Calais, France?

Mr. FRANCIS.—I do not know, sir, what they use.

EDWARD P. NORTH, Director Am. Soc. C. E.—There is one question I would like to ask about. Mr. Wilson speaks of a fill on the Bear River Canal 108 feet in height. There should be a pretty large allowance for

shrinkage there. What effort have they made to allow for shrinkage and how successful was that effort ?

Mr. Wilson.—There are several such fills on the line of the Bear River Canal. These fills are across drainage lines and the water which might flow down the drainage lines was carried through the fill in a culvert. Of course the deep fills would settle considerably. They were all allowed to rest about a year before the wooden flume which carried the water was laid on top of them. At the same time, subsidence will doubtless take place, and several of these fills have been seriously damaged by leakage from the flume. I think it is perhaps unusual to put a wooden flume on top of such a fill, the width of which hardly exceeds the width of the flume. I saw two of these flumes in February, when, after the water had been turned into the canal, the fill had been washed away and flumes and fills had all gone out. Piles were driven into these fills on which the flume rested, but the piles had not been driven to the solid ground surface and were not of much service. They are now caulking the flumes and hope to keep them in shape in that way.

Robert Moore, M. Am. Soc. C. E.—I don't know anything that better illustrates the need of good engineering and the damage done by bad engineering than some of the works that have just been described. One of them happens to have come within my notice, namely, the Arizona Canal. Some years ago I visited the head works of this canal, and found it to be an admirably conducted work, with the single exception of the diversion dam. This had been recently swept away and the whole work thereby rendered useless. The second dam constructed since then is certainly a mistake; both the place and the manner of its construction being faulty. The dam mentioned as the third is in the proper place and is the only one that should ever have been constructed. The whole work in this respect has been a series of mistakes. It is very strange that people will invest so much money and risk it upon such flimsy constructions as these. The western country is now infested with ditches that carry no water. If something can be done to correct this evil it will be of great value to the country.

Mr. Wilson.—I would like to say in defence of the engineer who designed the head works of the canal, when he found that I was going to look over them carefully, he told me the form of construction was one that he protested against, and had informed the company that it would be carried away, but the pressure brought to bear on him was such that he had to build it in a very short time and in that place. He would have placed it where it is now to be placed, where the head works of the canal would be good, and where the dam itself would have good abutments and foundation, but he could not build it there inside of eight months, and they told him it had to be built right away, so he built it right away, though he was well aware of its instability.

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INSTITUTED 1852.

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493.

(Vol. XXIV.—August, 1891.)

SOME EXPERIMENTS ON THE TRANSVERSE BREAKING STRAIN OF PLATE GLASS.

By G. W. PLYMPTON, M. Am. Soc. C. E.

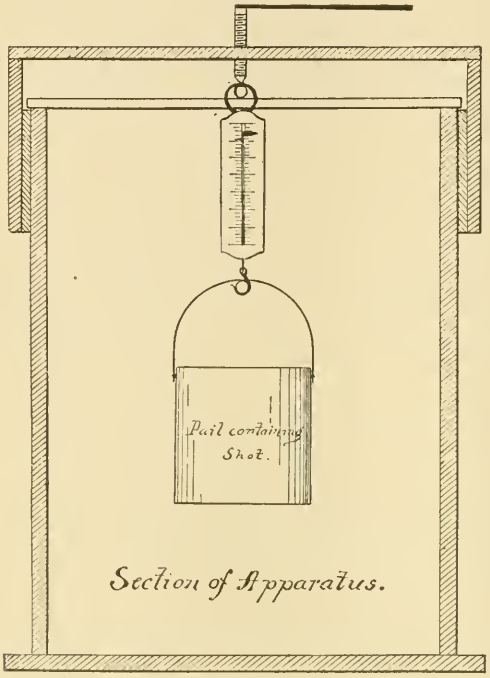
WITH DISCUSSION.

Having occasion some three months ago to determine the pressure which plate glass windows would safely withstand when exposed to strong winds, I consulted numerous tables to ascertain what had been recorded as determined transverse strength. Only two or three such records were found and these were discordant; naturally so from the nature of the material. I resolved therefore that I would subject specimens of the glass to be used in the structure to the test.

The specimens tried came from Holland. They were of a uniform length of 20 inches, and a width of 2 inches, and the thickness varied as shown in the table. Every strip was broken by weights added at the center from a relatively small weight up to that producing rupture. The strip or bar of glass rested on upright supports just 18 inches clear. A bar of half-inch round iron was placed across the middle of the test piece and projected far enough on each side to permit a spring balance to be hung on each end. A second bar of the same size was placed in the lower ring of the balances, and this bar supported a pail into which shot was poured for the trial weights.

To measure the deflections a wooden cross-bar was placed above the glass strip. Through the center of this bar a pointed screw of thirty-

two threads to the inch projected, and was of sufficient length to follow the deflections. This screw was run down at each measurement so as just to touch the bar on the middle of the glass strip. To insure regularity in the pressure of the screw upon the rod a strip of paper



Section of Apparatus.

Scale.
Inches.

was interposed to receive the pressure, and this was applied so as to permit the paper to be gently pulled between the screw point and the bar. The deflections were taken by counting the turns of the screw, the fractions being made apparent by a long horizontal indicator wire soldered to the head of the screw.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 1.....	.315	16	.035
.....		20	.043
.....		24	.053
.....		28	.059
.....		32	.066
.....		36	.074

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 1.....		40	.082
.....		44	.090
.....		46	Broke
Experiment No. 2.....	.315	16	.037
.....		20	.045
.....		24	.051
.....		28	.059
.....		32	.066
.....		36	.074
.....		40	.082
.....	.315	44	.090
.....		48	.098
.....		49	Broke
Experiment No. 3.....	.345	16	.027
.....		24	.049
.....		32	.058
.....		40	.062
.....		47	Broke
Experiment No. 4.....	.335	20	.043
.....		30	.059

Left under this stress broke in four hours twenty minutes.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 5.....	.295	8	.027
.....		16	.051
.....		24	.074
.....		31	Broke
Experiment No. 6.....	.290	8	.030
.....		16	.051
.....		24	.074
.....		28	Broke

Experiment No. 7, with a thickness of .290. This strip was loaded with 20 pounds and left undisturbed fifty-two hours. The weight being removed, the strip recovered its straightness completely.

	Thickness, inch.	Weight, lbs.	Deflection, inch.
Experiment No. 8.....	.320	16	.043
.....		24	.058
.....		30	Broke in one minute.
Experiment No. 9.....	.300	16	.043
.....		24	.056
.....		30	Broke after five minutes.

Experiment No. 10 was the same strip as No. 7. After the first test the load was increased to 30 pounds, after five minutes to 36 pounds, and after eight minutes to 38 pounds, when it broke immediately.

The regular increase of deflection seems to show no limit of elasticity below the breaking weight. Tests with the polariscope verified this conclusion. No. 7 had suffered no molecular change after a continuous strain of more than half its breaking weight for fifty-two hours.

If C be called the co-efficient of rupture in the formula $W = \frac{cbd^2}{l}$ Then in experiment No. 1, 4 176 pounds; in No. 2, 4 452 pounds; in No. 3, 3 552 pounds; in No. 4, 2 412 pounds; in No. 5, 3 204 pounds; in No. 6, 3 000 pounds; in Nos. 7 and 10, 4 068 pounds; in No. 8, 2 640 pounds; and in No. 9, 3 000 pounds.

If the formula $\frac{1}{8} Wl = \frac{1}{6} Rbd^2$ be used, employing the assumption that the neutral plane or axis is at half the depth, then R, or the rupturing stress per unit of section, would in each of the above cases $= \frac{3}{4} c$. Trautwine's formula, deduced from the Millville experiments, gives for the breaking weight $W = \frac{2\,000\ bd^2}{l}$, which is a somewhat lower value than would be deduced from the above results.

DISCUSSION.

JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—In comparing Mr. Plympton's results with that obtained by my father in July, 1868, with Millville flooring glass, and quoted by Mr. Plympton in his concluding paragraph, it is important to take into consideration the effect of the time during which the test-piece is subjected to its load. The values of the co-efficient C obtained from Mr. Plympton's experiments are as follows:

No. of Experiment.	Co-efficient: $C = \frac{\text{load x span}}{\text{depth}^2 \text{ x breadth.}}$	Time Under Final Load.
1	4172	
2	4444 (maximum).	
3	3554	
4	2406 (minimum).	4 hours 20 minutes.
5	3206	
6	2996	
7	....	
8	2637	1 minute.
9	3000	5 minutes.
10	4067	

The slight differences between Mr. Plympton's values of C and my own, as given in the above table (both deduced from his experiments),

are doubtless due to his neglecting final decimals in the quantity d^2. In experiment No. 7 the bar was loaded with 20 pounds and left undisturbed for fifty-two hours. It received no permanent set, and was again used in experiment No. 10.

It will be noticed that in the three experiments (Nos. 4, 8 and 9) in which the time under load is stated, the value of C is distinctly lower than in any of the others (in which we may assume that rupture immediately followed the application of the final load), except No. 6, which, while the least of these, is practically equal to the greatest of the time tests; and the least value of C in the whole series (2406) is given by experiment No. 4, where the time under load was greatest (four hours twenty minutes). The mean value of C for the three-time tests (experiments Nos. 4, 8 and 9) is 2681, while that for the six others (Nos. 1, 2, 3, 5, 6 and 10) is 3740, or 39.5 per cent. greater. Now, by reference to my father's notebook, I find that his test-piece, which gives $C = 12 \times 170 = 2040$, "bore load for nearly one minute before breaking," and his experiment would therefore be comparable with Mr. Plympton's Nos. 4, 8 and 9 (mean $C = 2681$) rather than with the others (mean $C = 3740$), and would thus make a better showing for the Millville flooring glass of that early day, as compared with the modern Dutch plate window glass, than would appear from a comparison with Mr. Plympton's results as a whole.

It would have been interesting if Mr. Plympton had proceeded to explain the application of his valuable results to the actual problem in hand, viz., "the pressure which plate glass windows would safely withstand when exposed to strong winds." In that case we have a rectangular plate, fastened along all four (?) of its edges and subjected to a load (approximately) uniform, but quite indeterminate and beyond our control; while in the tests we have a beam merely supported at its two ends and bearing an accurately known center load. Will Mr. Plympton give us the steps by which the safe thickness in the former case is deduced from the breaking load in the latter?

Prof. G. W. Plympton, M. Am. Soc. C. E.—In reply to Mr. Trautwine's remarks, I have only to say that he is quite right in assuming that in the majority of the experiments the rupture followed immediately upon the application of the final increment of load.

In reply to the query regarding the application of these results to the solution of the final problem (viz., the wind pressure which plate glass windows will safely resist), it may be said that nothing more is to be claimed for these results than that a safe limit for a known distributed pressure is fairly well indicated, at least when supported at opposite sides (or ends). How much this estimated thickness can be reduced for rectangular plates supported or held in water-tight fastenings along all the edges, can only be properly determined by experiments tried under such conditions.

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(Vol. XXV.—August, 1891.)

A CHEAP COVERED RESERVOIR.

By ARTHUR D. FOOTE, M. Am. Soc. C. E.

WITH DISCUSSION.

The domestic cold water supply for Boise City, Idaho, is obtained from a number of shallow artesian wells sunk in the hollow of the flank of the mountain which rises north of the town. The words domestic and cold are used advisedly as the town is supplied with water for irrigation purposes by canals from the river, and hot water from artesian wells a short distance east of the town will, it is said, soon be distributed by pipes throughout the city. The cold water wells overflow at an elevation of about 175 feet above the town, and at present supply about 1 000 000 gallons per day. The formation of the country in their immediate vicinity is a soft sandstone, lying horizontally, with occasional strata of talc and hard shale. The surface has the usual ridges and hollows of a rapidly sloping mountain side. Several small tunnels have been driven into the hill, in this vicinity, in a vain search for coal. An examination of the material encountered in running these tunnels suggested to me the idea of excavating a reservoir in the rock. As a preliminary experiment the sides and roof of one of the tunnels were plastered with cement mortar, which remained perfectly sound after three months.

A point was selected on the side of one of the ridges forming the gulch in which the wells are located, and slightly below them, and an open cut 8 feet wide was driven horizontally into the hill until the face of the cut was about 12 feet high. A stratum of stiff pipe-clay about 3 feet thick showed across the face of the cut, the bottom of which was 7 feet above the flow of the cut. This material was so hard and solid that it was decided to use it for the roof of the chamber, instead of excavating an arched roof, as first intended. A gallery 6 feet wide and 7 feet high was driven 10 feet, and then widened 7 feet on each side, and continued inward until there had been excavated a chamber 125 feet long, 20 feet wide, and 7 feet high. The material encountered was sandstone, easily and rapidly broken down with a pick, and was removed with the ordinary grading dump carts and horses, at a cost, including haul, of about thirty-five cents per yard.

No crack or sign of settlement or movement of any kind appeared during the excavating. It was thought advisable, however, by the directors to use every precaution possible against accident; accordingly a row of brick pillars, 8 feet from center to center, extending lengthwise through the middle of the chamber, were put in. These pillars were 20 inches square, set on flat stones 3½ feet square, with cap-stones of the same size. The cap-stones were covered with a layer of cement mortar and forced up against the roof with jacks and then the brick piers built up to them as solidly as possible. After the completion of the pillars the inside of the chamber was carefully plastered with cement mortar composed of three parts sand to one of Portland cement. The walls and roof were covered about half an inch thick and the floor about 1½ inches. A brick wall, in which were built the inlet and outlet pipes and a ventilating manhole, closed the entrance, and the water was admitted. This was about four months ago, and as yet there has not been the slightest sign of cracking or scaling of the cement, nor has there been any movement or settlement whatever in or around the structure.

This reservoir is a small affair, and was largely experimental, but the result has been that the company will now, as occasion requires, construct similar chambers on either side of the present one, greatly increasing the length, but not increasing the span of the roof. It is evident that these chambers will be much cheaper, will require less repairs, and will keep the water in better condition than any structure that could be

built outside. Of course, it is well known there are many reservoirs or "cisterns hewn out of the rock" in the far East, but I think they are somewhat uncommon in this country, and I send this description thinking it may be suggestive to any member of the Society who may happen upon similar conditions.

DISCUSSION.

A. Fteley, Vice-President Am. Soc. C. E.—The subject is an interesting one. It seems to me it should lead to some remarks from the gentlemen interested in storage of water. I might say in connection with it that a short while ago as I was making inquiries as to the influence of covered reservoirs on the purity of water supplies, I came across one fact of some interest. An engineer in California was very much troubled by an extensive growth of algæ, and having read that when the reservoirs were covered the water could be kept free from this growth, he conceived the idea of surrounding the part of that reservoir near the intake with a large floating platform. As a result he found that although there was a constant motion of the water from the outside to the portion which was covered, yet the water supplied was very much improved. This information came from a reliable source, and I thought the incident might have some interest, as I never heard of it before.

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INSTITUTED 1852.

TRANSACTIONS.

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495.

(Vol. XXV.—August, 1886.)

A SIMPLE DIAGRAM, GIVING, BY INSPECTION, THE DIMENSION OF WOODEN BEAMS FOR A GIVEN SPAN AND LOAD.

By J. M. MICHAELSON (Student).
Presented by E. A. FUERTES, M. Am. Soc. C. E.

The frequent need of designing beams within the limits of the commercial sizes of timber makes the following diagram devised by the author particularly useful, as it gives at a glance, within wide limits, all the dimensions of beams which will safely bear a given load over a given span. It is the graphical expression of the equation for safe loading for a stress of 900 pounds per square inch at the outer fibers, or for any other safe load.

Before giving the construction and proof of the diagram, its use will be illustrated by an example. Let the given span be 12 feet, and the load a uniform one of 1400 pounds per foot.

On the line AC there are given a series of figures which represent the loads in pounds per lineal foot of beam, divided by 100; and also widths of beams in inches. On the inclined line AB are given spans in feet. To apply the diagram we first find the point d where a load line, $14d$, parallel to the line AB intersects the span line $12c$ drawn from C through the span 12 feet on line AB. Now this line de intersects two sets of lines parallel respectively to AB and to AC, and any span line

and load line intersecting on *de* or below it will indicate dimensions of a stick which will safely bear the load. For example, we find 8×16 inches intersecting in *f*, also 9×15, 12×13, 10×15, 11×14, etc.

As a general rule, of course, the deeper a beam the more economical it is, but the selection may be governed by the method of support, the provisions against buckling, market sizes, etc.

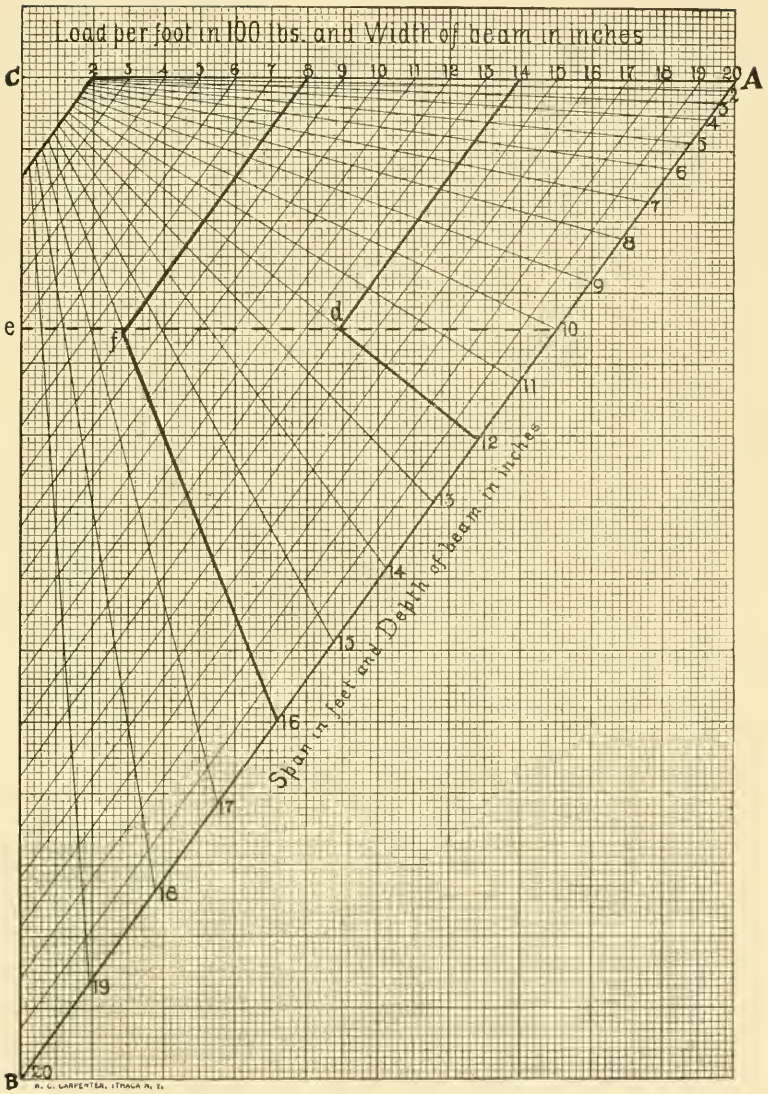
The foregoing example involved a uniformly distributed load; but the diagram may be used for any manner of loading and for any safe stress ; for a uniform load may be found which will produce a maximum stress equal to that produced by any other manner of loading. Thus for a load in the center of the span, the load taken in the diagram would be double the actual load ; for a uniform load on a cantilever four times, and for one on the end of a cantilever eight times the actual. For more complicated cases of loading the method of moments is to be used.

It will be seen later from the equation for safe loading, that if any other stress than 900 pounds per square inch is used, it is equivalent to multiplying the load per foot by the ratio of 900 to the assumed stress; *e. g.*, for a stress of 1 200 pounds, the load per foot is multiplied by three-quarters, and the diagram used as before. In other words, if we increase the allowable stress on a stick supporting a given load, we can employ a lesser value of the product bh^2 given by the formula for the amount of the internal forces.

The construction is simplified by the fact that no moments of any kind are computed. On the line AB, the distances $A2$, $A3$, $A4$, ..., are taken respectively $= 4$, 9, 16, ..., *i. e.*, $A2 = 2^2$, $A3 = 3^2$, $A4 = 4^2$..... These points are then joined to any point, C; and lines are drawn parallel with AB, dividing AC into any convenient number of parts. To facilitate explanations, let the lines converging in C be called span and depth lines, those parallel with AB, load and width lines, and those parallel with AC, moment lines. For a beam, with simple end supports, bearing a uniform load of w pounds per foot over a span of l feet, the maximum moment in foot-pounds is $M = \frac{1}{8} WL^2$.

The load remaining constant, the maximum moment varies directly with the square of the span, and since the distances $A1$, $A2$, $A3$, etc., on the line AB are made proportional in length to the squares of the numbers 1, 2, 3, etc., they represent graphically the moments of a constant load.

PLATE XLI.
TRANS. AM. SOC. C. E.
VOL. XXV, NO. 495.
MICHAELSON ON DIMENSIONS OF WOODEN BEAMS.



Again, if the span remains constant, the maximum moment varies directly with the load per foot. Therefore, for a given span and load, the maximum moment is represented by the part of the load line intercepted by the line AC, and the intersection of the given span and load lines, *e. g.*, $A12$ on AB represents the maximum moment for a span of 12 feet and a load of 2 000 pounds per foot; while $14d$, the portion of the load line 1 400 intercepted between AC, and the intersection of the span line 12 with the load line 1 400 represent the maximum moment for a span of 12 feet and a load of 1 400 pounds per foot; for, from similar triangles, $14d : A12 : : 14 : 20$, *i. e.*, $14d$ and $A12$ are proportional to the loads 1 400 and 2 000, and the span remains the same. It is then evident that all span and load lines intersecting in the same moment line have the same maximum moment.

The well known formula for the moment of the internal forces in a homogeneous prismatic beam, rectangular in cross-section is $M = \frac{1}{6} pbh^2$, b representing the width and h the the depth of the beam. If now p equals a safe stress of 900 pounds per square inch, and b and h are in inches, the maximum moment for safety $= 150\ bh^2$ inch pounds; and if w is the uniform load per foot in pounds, and l the span in feet, the equation for safe loading is

$$150\ bh^2 = 12\ (\tfrac{1}{8}\ wl^2) = 150 \left(\frac{w}{100}\right) l^2.$$

Thus for a stress of 900 pounds per square inch, if the load per foot be expressed in units of 100 pounds each, the span in feet, and the width and depth in inches, then the moments of the external and internal stresses have the same constant, 150; and if $b = \frac{w}{100}$, then $h = l$; and conversely. Hence the equation for safe loading is satisfied by a width and depth in inches equal numerically to the load per foot in units of 100 pounds, and the span in feet respectively. But, as shown, all width and length lines intersecting in the same moment line have the same moment. Therefore, the moment line determined by the intersection of a given span and load line contains the intersections of all width and depth lines which satisfy the equation for safe loading.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

496.

(Vol. XXV.—August, 1891.)

THE ADVANTAGES OF A LONGITUDINAL BEARING SYSTEM FOR RAILWAY TRACKS.

By THOMAS C. CLARKE, M. Am. Soc. C. E.

It is now admitted by civil engineers and other railway experts, that our American system of track which has done so well in the past, owing to its strength, elasticity, and economy of first cost and maintenance, is now in a bad way, and must be modified in some manner to meet the ever-increasing weight of cars and engines. This has grown during the last fifteen years from 5 tons on a locomotive driver to 9½ tons; and 3 tons per car wheel to 5½ tons; and the end is not yet. Our track is giving out at all points. Joints which did well under lighter loads, now sink and allow rail ends to be battered down. Steel rails are not hard enough. Their heads crush, or the material itself flows under the heavy wheel loads.

A new danger has shown itself. The old form of spike can no longer hold the rails to the ties. The sinking at the joints makes the rails saw up and down, and loosens the spikes. Then the gauge spreads, and the trains leave the track. This form of accident is becoming more alarmingly frequent every year.

The supply of timber for ties is becoming a serious question. Statistics show that from 60 000 000 to 70 000 000 of wooden ties are yearly required in the United States. Chemical preparation of the timber is claimed to double the life of the tie, so far as decay is concerned, at an increase of only about one-third of the first cost. But a new trouble appears. The great weights on the rails, aided by the sawing motion above described, cut their bases into the ties, and destroy them sooner than they rot.

Thus we have rails, fastenings and ties, all failing together. What is the remedy? Mr. Whittemore, of the M. and St. P. R. R., proposes to increase the bearing surface of rails by making the heads flat and increasing their width. Mr. Sandberg, of London, sees no other way but to increase all the dimensions of the present rail and its fastenings, until his new "Goliath" pattern weighs 100 pounds per yard. Improved forms of joints—notably the Fisher, Cloud & Thompson, so called from their inventors, have met with considerable favor.

Cutting into the ties can be prevented by the use of metallic tie-plates. This, while excellent in itself, complicates the problem of fastening the rails and ties together. In England, the cast-iron chair on every tie is a good but expensive remedy.

Various improved forms of spikes and bolts have been devised to hold the rails and ties more firmly together. The most radical change, however, is the adoption of steel cross-ties, originally devised for countries where timber was scarce, or where wooden ties were eaten by insects. They have spread all over Europe, and are now being used experimentally on some of our larger railways. Besides their durability, they have the merit of allowing the rails to be firmly fastened to them and spreading of rails is prevented. But the sinking of the rail joints still causes the rail to saw up and down. As it cannot leave the tie, they both move together, and after a time get loose in the ballast. Also, the tie, being but a shallow piece of channel iron with flanges 2 to $3\frac{1}{2}$ inches deep, has no stiffness crosswise. The wheel is only supported by a short length of the tie, and they bend down under the rails, assuming a shape with two bends downward and one upward in the middle. (See Fig. 1, Plate LII.) This works the ties loose in the ballast, and from these causes, it is difficult to keep a metallic cross-tie system in good line and level.

From this smallness of the bearing surface, the metallic cross-tie track, under American wheel loads at least, not only sinks at the joints

but at each tie as it is passed over, making an uneven track longitudinally. There is also a serious complaint of the want of elasticity in this form of track. The first cost of these ties is considerable, as will be seen from the comparative table given hereafter. Various attempts have been made to improve them, by inserting wooden blocks, etc. This only leads to complication of parts and increased cost, while it does not remedy the vital defect of weakness of joints, and the consequent vertical movement of the rails.

On considering all these matters, and personally examining a great many improved forms of track, it seems to the author that the only radical cure of the difficulty is to return to the old form of continuous rail and continuous bearing, improved to avoid those defects which experience has shown, and made capable of being extended in dimensions and modified in material to meet the ever-increasing loads of the future.

The defects of the continuous-bearing system as formerly made are as follows: The early form of continuous rails was made of such soft iron that the bearing parts cut into each other and soon worked loose. This cannot occur with steel members of proper proportions. The rails rested on longitudinal timbers. These had to go down so deep that they cut off drainage between the rails. Also, it was difficult to unite them to the wooden cross-ties necessary to preserve the gauge. The great number of joints caused decay. For all these reasons the continuous system, although giving a very smooth track while new, soon got out of order, and was replaced everywhere by the wooden cross-tie system, which was less expensive to make and far less so to maintain.

The radical, and as the author believes, irremediable defect of the cross-tie system is its low joints. The rail itself is treated as a continuous girder resting on yielding supports and spliced at the joints. If the attempt is made to splice the joint within the rail depth, as by angle bars, it deflects on account of want of depth. As the joint goes down the middle of the rail goes up. If we strengthen the joint by putting pieces below it, intended to be in tension, while the rail is in compression, thus converting it into a deeper form of girder, the necessity of allowing for expansion and contraction makes it necessary to use oval holes for the bolts. These allow the bolts to move longitudinally, and thus they do not resist the shearing strains, and hence do not prevent deflection.

This form of joint works well while new, but after a while gets loose like the others, and low joints are found. The only way, in the author's belief, to absolutely prevent low joints is to place continuous bearing pieces under the whole length of the rails and breaking joint with them. The solid part of the bearer supports the joint of the rail, and the solid part of the rail bridges over the joint of the bearer, and what is of the highest significance, a small amount of longitudinal movement or play between the upper and lower members will not cause deflection, when arranged as described. The requisites of a properly designed longitudinal system are as follows:

First.—The first is, that the longitudinal bearer under the rail shall be stiff enough to transmit the load to such a distance, on each side of the wheel, as will limit the pressure to not much over 2 tons per square foot of bearing surface, without requiring excessive width. Experience has shown that a greater pressure than 2 tons per square foot will sink ties too deep into the gravel or broken stone.

Second.—The next thing is to attach the rails and bearers together by a form of fastening, strong enough to resist all strains and shocks, and yet allow of freedom of the rail to expand and contract, independently of its bearer. It must also be held to its bearer, so that creeping of the rail on the bearer may be prevented, and that without any notching or cutting of the rail that will impair its strength. The rails must break joint with the bearers. The fastenings must be so made that the rails can be quickly removed and replaced by new ones without disturbing the bearers. The fastenings must be able to hold for a time a broken rail, so that it will safely pass the trains, and no system but the longitudinal can do this.

Third.—The bearers (and rails) should be united firmly together by light metallic gauge ties, placed near enough to properly preserve the accuracy of the gauge.

Fourth.—The bearers and gauge ties should be of such shapes as can be easily tamped with gravel or broken stone; as will stay in place vertically, laterally and longitudinally, and will allow of drainage to pass between them.

Fifth.—The system should be so planned that no difficulty of construction can occur at curves, either in alignment or elevation of outer rail. Also, it should be so made as to easily join to the ordinary form of T-rail at turnouts and switches.

Sixth.—Besides the obvious advantages which such a construction gives, there are two others: The upper rail can be made of a harder and better worked steel, while the bearer can be made of a softer and tougher quality of metal. Probably basic steel would do for this.

Owing to the rails being supported under their entire length by continuous bearers, they can be made of less depth and sectional area in their lower flanges than at present. The metal so saved can be put into the head of the rail, where it is most needed.

It is believed that rails can be designed for a longitudinal system with heads 3 inches wide, and instead of weighing 110 pounds to the yard, as in the last sheet of sections given by our Committee on forms of rails, they need not weigh over 70 pounds to the yard. This saving of metal can be applied to reducing the cost of the whole system. The wear being confined to the upper rail, the amount of metal which goes into the scrap heap is the least possible. Longitudinal bearing systems are not new, nor entirely experimental—as may be seen by reference to the report of E. E. R. Tratman, Assoc. Am. Soc. C. E., to the Department of Agriculture. This system has been in use on the Austrian State Railways, Northwestern Division, for eleven years. It is the invention of Mr. Hohenegger, Chief Engineer, from whose report, quoted by Mr. Tratman, the following description is condensed. (See Figs. 2 and 3, Plate LXII.)

There are 59½ miles, of which 37.21 miles are tangents, and 22.30 miles curves varying from 9 840 to 935 feet radius. There are daily, both ways, two fast express, two express, two slow passenger and ten freight trains. Passenger engines weigh 42 tons with 6.2 tons on driving wheels. Freight, 45 tons with 5.67 tons on driving wheels. Only one longitudinal bearer has been renewed. The only wear observed has been where, in some cases, the flange of the inner rail on curves has worn slightly into the bearer. The longitudinals, like the rails, are laid just as they come from the rolls, but no rusting at all has taken place. The track is reputed to stand admirably and requires little labor for maintenance, either on tangents or curves. There is no trouble with the fastening of rails to the bearers, and the nuts being held in position by nut-locks, tightening of the nuts is rarely necessary.

The fastening of the rails to the bearers prevents creeping of the rail on the longitudinal by the firm hold of the clamp on the rail flange and on the rib of the longitudinal. The inclined outer face of the clamp admits of

an adjustment of the gauge by slacking one nut which allows the clamp to rise and move back. The rail is then shifted, the opposite clamp pushed down to fit and both nuts screwed tight. The rail flange butts against a boss on the lower side of the clamp which receives the force of the lateral thrust from passing trains, and transfers it to the rib of the longitudinal, thus protecting the bolt from wear. As there are no spikes to be driven, a smaller gang can keep the track in order. In consequence of the continuous support of the rails no rails nor splice plates have ever broken. The rail weighs only 64.30 pounds per yard, while a rail on cross-ties would have required a weight of 100.43 pounds per yard.

Obviously there are two defects in this design of track. The form of the longitudinal is such that Mr. Hohenegger tells us that the ballast which consists of broken stone and river gravel, is compressed by tamping and the pressure of passing trains, to such an extent that it can only be loosened with a pick. This compression extends a foot below the longitudinal and prevents the quick drainage of water from between the rails. This could have been prevented by a better section of bearer, possibly of a T shape. The ballast could then be tamped against the vertical member from both sides and there being nothing to confine it there, the packing would not have taken place. Another mistake was to make the joints of the rails and of the longitudinal bearers coincide, instead of breaking joints. Mr. Hohenegger says nothing about the existence of low or battered joints, but they must have existed, and breaking joints with the bearers would have prevented them.

A comparison of the cost of the cross-tie systems now in use, with that of a continuous bearing system is as follows, prices of material being alike :

First.—Wooden cross-tie system and 60-pound rails as used on local lines.

	Mile of Single Track.
106 tons rails at $35	$3 710
2 640 ties at 55 cents	1 452
352 splice bars and bolts	440
Spikes	130
Laying track	500
	$6 232

Second.—Improved system with 80-pound rails, treated ties, tie-plates, and two pairs interlocking bolts per rail, as used on Trunk lines.

140 tons rails at $35	$4 900
2 816 treated ties at 80 cents	2 253
352 tie plates at 10 cents	35
352 long splice bars and bolts	720
Interlocking bolts	231
Laying track	600
	$8 739

Third.—System further improved by use of 85-pound rails, and steel cross-ties.

149 tons rails at $35	$5 215
2 464 steel ties and fastenings at $3	7 392
352 long splice bars and bolts	720
Laying track	700
	$14 025

Fourth.—Hohenegger system of longitudinal bearings used on Austrian State Railways. Rails 64.38 pounds per yard.

113.4 tons rails at $35	$3 919
105 tons longitudinals at $46	4 835
32 tons, cross-ties, plates, splice bars and bolts at $50	1 600
Laying track	800
	$11 154

From this it would seem that a first-class track, made with metallic longitudinal bearers, need not exceed the cost of a first-class track laid with metallic cross-ties.

A letter and drawings received from Herr Hohenegger are appended.

APPENDIX.

In answer to your esteemed query, dated June 17th, I beg to give the following information, taking reference to the "Advance Copy" received from you.

Article 1. I have used in my system of superstructure with longitudinal bearers that is discussed by you, both the combined joint as represented in your sketch and the broken joint.

From the drawings embodied, you will understand the construction of the broken joint, in which the joints of tie and rail do not coincide. The combined joint in which rail and longitudinal bearer butt against each other on the mutual saddle-shaped splice, *S*, facilitates the laying of the superstructure somewhat, and gives generally good results, because the carrying capacity of rail angle bars and of tie splice equals about that of rail and of longitudinal bearer; the broken joint, however, seems, in the length of time, to offer more resistance against the depression of the joints. I have tried both systems from the beginning, in order to gather experience and to reinforce, if necessary, those parts of the joints which should prove too weak. Obvious weaknesses in this regard have not been noticed up to the present time, but I would now give a larger bearing surface to the tie-saddles.

Article 2. The fastening of the rails on the longitudinal bearers by means of wedge-shaped tightening plates, which bear against wedge-shaped ribs of the longitudinal bearers, proves eminently satisfactory, with the only alteration, that we used two-sided unsymmetrically formed tightening plates instead of the one-sided ones, to facilitate the obtaining of the correct gauge where the cross-ties are not well punched or where they are bent as in curves.

The wedge-shaped form of the tightening plates has done away entirely with the very undesirable rusting together of the bolts with the bearers. The exchanging of worn-out or injured rails is effected very easily without any disturbance whatsoever of the position and of the fastenings of the longitudinal bearer.

Within the fourteen years of the existence of this superstructure with longitudinal bearers, two cases only of breakage of rails have occurred, and in these several trains passed without derailment over the remnants of the rails that had been broken in many pieces. In both cases blow-holes caused the heads of the Bessemer steel rails to be shattered in many small pieces, so that the wheels ran without being derailed, partly on the rail-web, partly on the longitudinal bearer, though one of the broken pieces belonged to the outer rail of a rather short curve.

Article 3. I have never introduced gauging rods proper, as they are used in the superstructure with longitudinal bearers of the German rail-

ways, but have been satisfied with one transverse angle for each 9.84 feet (3 meters) of bearers fastened to the lower side of the bearers with short pieces on saddles, *s*. This transverse connection has perfectly sufficed to maintain the gauge; if it should not prove sufficient more cross-connections would be put in place without great expense, but they have not been necessary so far.

Article 4. Longitudinal bearer, transverse angle-tie and tie-saddles are easily tamped with gravel when the superstructure is newly laid; in later years, after the box-shaped volume of ballast underneath of the longitudinal ties has consolidated into a mass of beton, the longitudinal bearers must be lifted off it and the beton-like old ballast has to be replenished by a layer of fine ballast. However, this work becomes necessary only very seldom because the superstructure with longitudinal bearers retains its position much better than if laid with transverse ties, which each give way separately.

The (lifting) raising of the superstructure when depressed, is more difficult with sunk iron transverse ties than with longitudinal bearers, for the following reason. It is not possible to raise a single transverse tie without unscrewing a pair of rails, whereas with longitudinal bearers rail and bearer can be lifted together, necessitating only the loosening of the rail angle bars.

Article 5. The T-shaped bearer, as suggested by you, was tried in 1873 by the "Rechts Rheinische Railroad," but has not proved a success, because the vertical web of the T tears asunder the box-shaped mass of beton, instead of holding it together and of gradually rendering it a unit. All longitudinal bearers and transverse ties shaped similarly have been discarded for some time as non-practical. In order to lay the curves of the superstructure, the longitudinal bearers have to be bent in the ironworks according to the requirements of the earth curve. The supervising engineer must have an accurate table that shows how the segments follow each other, and he must lay his longitudinal bearers consecutively and in accordance with it. The laying is done very rapidly if this rule be observed. Up to 984 feet (300 m.) of superstructure are put down per diem on track where frequent trains are running. That is twice as much as could be exchanged of superstructure with transverse ties. The superelevation of the outer rail does not offer any known difficulties. The connection with cross-overs (Weichenstrasse) and switches, is not more difficult than in superstructure with transverse ties, if rail angle bars of equal section are used and if the cross-overs are put down on transverse ties. We do not have cross-overs on longitudinal bearers.

Article 6. The rails on longitudinal bearers are essentially lighter than those on transverse ties. They weigh only 18.106 pounds per foot (27 kg. per meter), against 22.132 pounds per foot (33 kg. per meter). Transverse fractures of rails have not yet occurred, those two cases excepted where the rails were destroyed through blowholes of the ingots.

FIG. Nº 1.
BENDING OF METALLIC CROSS-TIE. UNDER TRAIN.

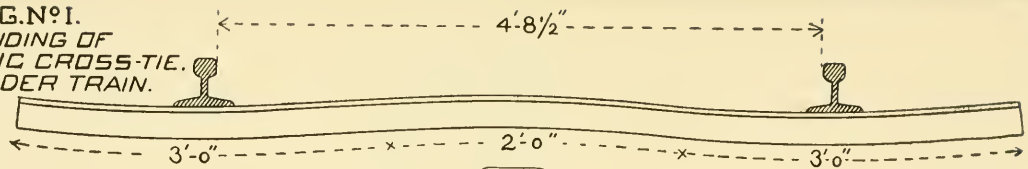


FIG. Nº 2.
CROSS SECTION OF FIG. 3 AT A.B. HOHENEGGER SYSTEM

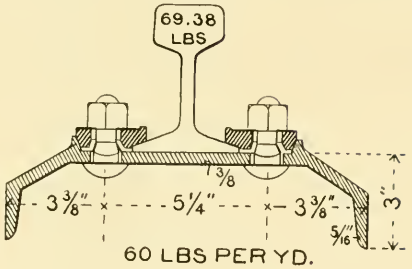


FIG. Nº 3.
PLAN.
DO. DO.

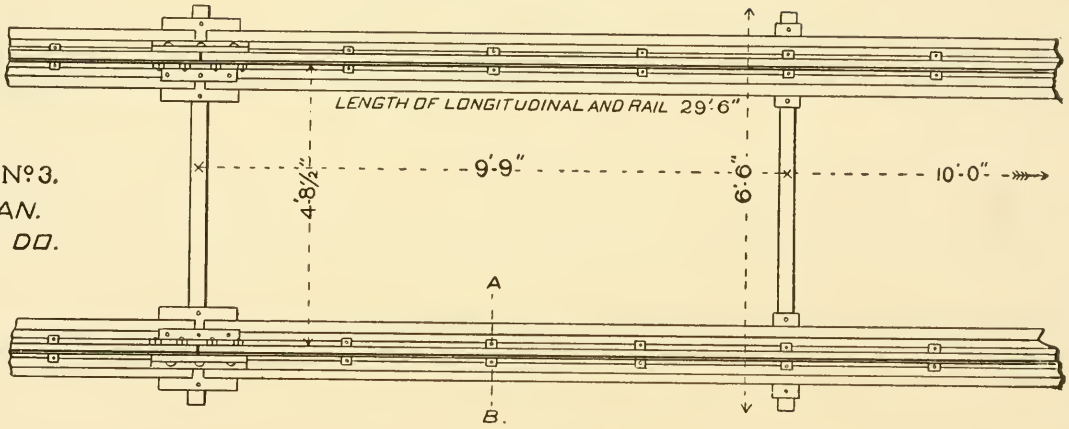
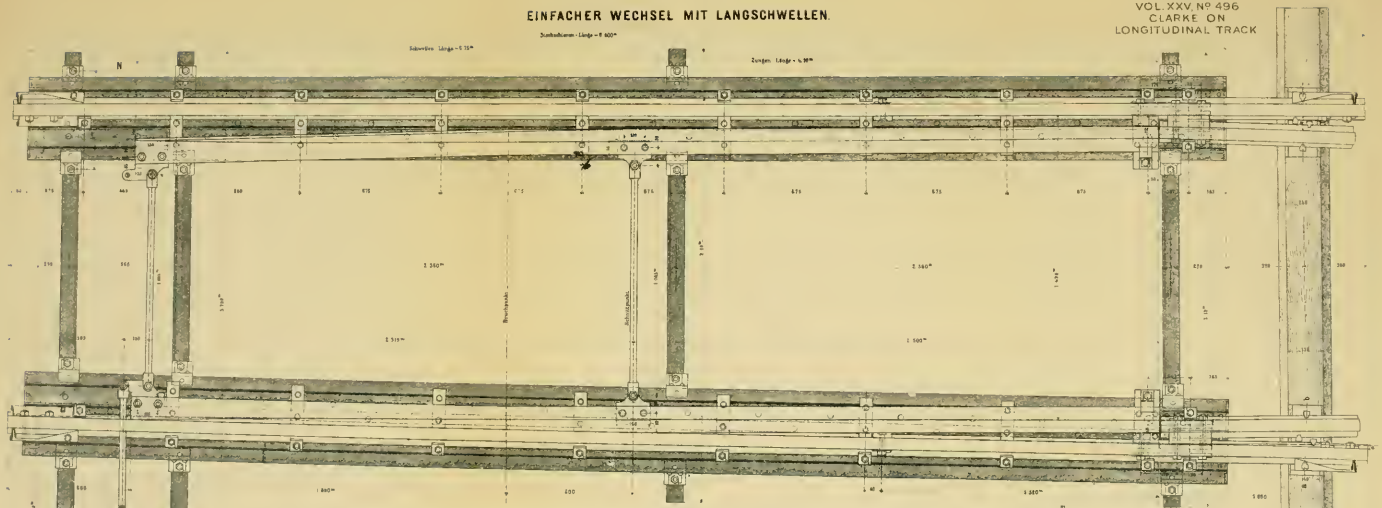


PLATE XLII.
TRANS. AM. SOC. C. E.
VOL. XXV, NO. 496.
CLARKE ON BEARING FOR RAILWAY TRACKS.

TRANS AM SOC. CIV. ENG'RS
VOL. XXV, Nº 496
CLARKE ON
LONGITUDINAL TRACK

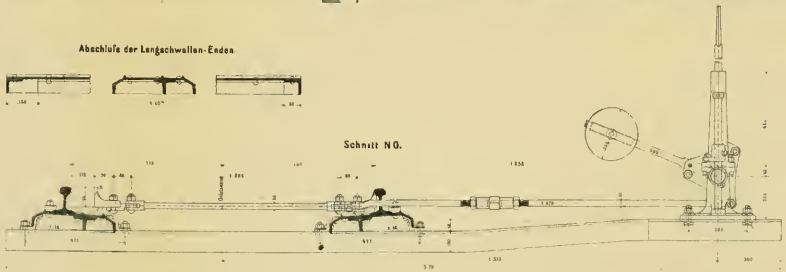
OESTERREICHISCHE NORDWESTBAHN

EINFACHER WECHSEL MIT LANGSCHWELLEN.

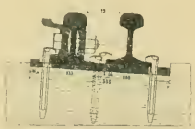


Abschluss der Langschwellen-Enden.

Schnitt N O.



Schnitt a b.



1:5

1:10

Genehmigt vom General Director
G. Gross.

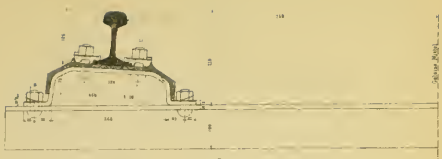
Wien im November 1867.
Ausgeführt im Baubüreau
W. Hohenegger.

OESTERREICHISCHE NORDWESTBAHN

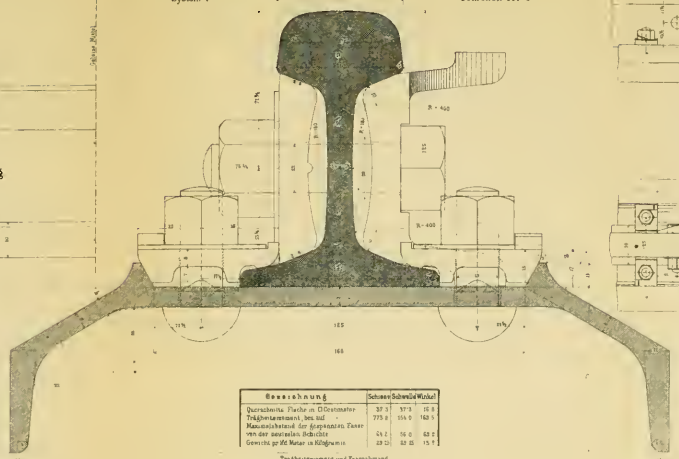
EISERNER OBERBAU MIT LANGSCHWELLEN AUS FLUSSEISEN.

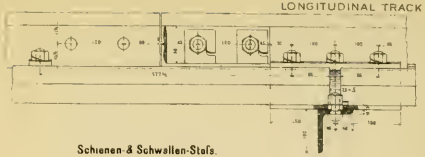
System V

Schienen Prf 6

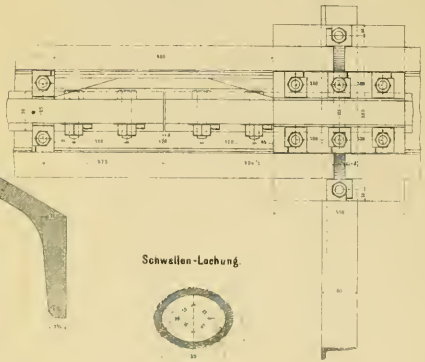


Querverbindung





Schienen-& Schwellen-Stofs.



Schwellen-Lachung.

Materialbedarf.

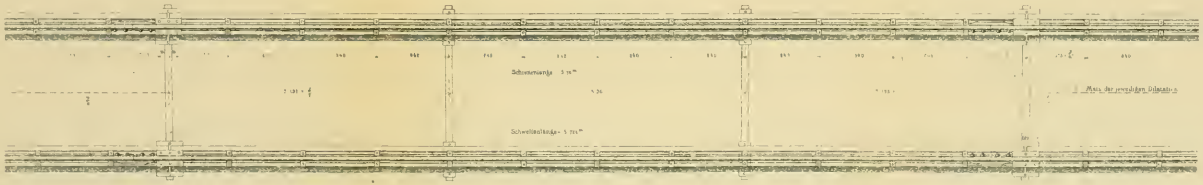
Stück	Gegenstand	Gewicht in Kilogram. Stück	zus.	Stück	Gegenstand	Gewicht in Kilogram. Stück	zus.
2	Stofsschienen à 8 m lg.	174	348	44	Schraffelagenkeil	0 80	3 60
4	Fufsschwellen à 2 m lg.	124	496	88	Schienennägel	2 15	19 19
2	Zwei Stofsschwellen à 2 m lg.	Da wie	79 80	176	Schwellenbolzen	0 35	6 40
1	Schwellenschwellen 50° lg.	0 70	35	352	Schienenbolzen	0 30	10 60
1	Gelenk-Lasch à 150° lg.	4 70	18 80	176	Querverbindungen	0 85	1 48
1	Winkelstofsbolzen	9 17	18 34	8	Schienenlagerplatten	0 68	2 04
8	Innenschrauben	0 90	7 20	44	dto.	0 15	0 72
8	Laschenschrauben	2 40	19 20		Eine geschlossene Geleise		1324 00

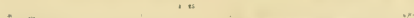
Gewicht per Mtr Meter Geleise= 135 8 Kilogr

Bezeichnung	Schiene	Schwelle	Winkel
Querschnitts Fläche in Centimeter	37 3	37 3	16 8
Trägheitsmoment, bei mit	775 8	154 0	163 5
Maximalabstand der äufsersten Faser von der neutralen Schichte	4 4	3 6	6 3
Gewicht per Mtr in Kilogramm	29 13	29 13	13 1

Trägheitsmoment und Faserabstand mit Berücksichtigung der Lochung.

Geleisenlage.





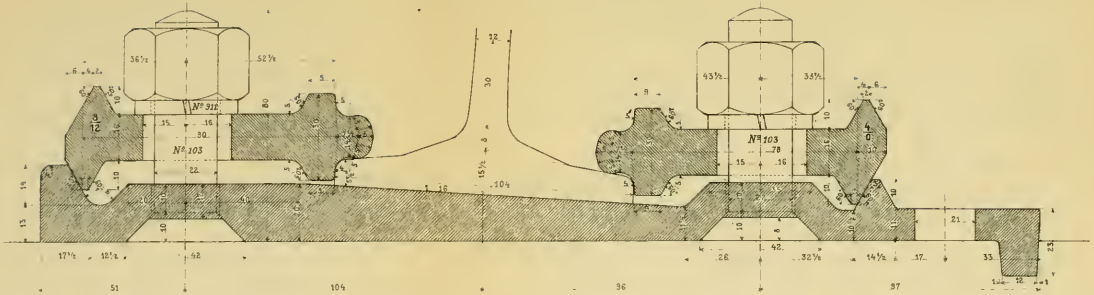
Lith. Anst. v. Steinitz, Prag
C. Graus

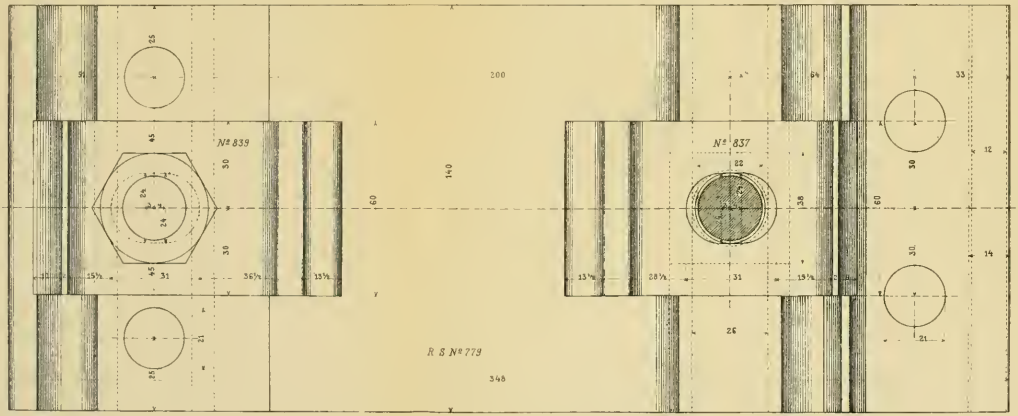
Wien
Verlag v. Bauindustr.
W. Hohensegger.

OESTERREICHISCHE NORDWESTBAHN

UNTERLAGS - SPANNPLATTEN

für Holzschwellen - Oberbau.





The rails are, as a matter of course, made of harder steel, whilst the longitudinal bearers are manufactured from soft Gilchrist—Thomas metal.

Up to date, 55.89 miles (90 km.) about, of superstructure with longitudinal ties have been laid and have given satisfaction in every respect. The quicker extension of this system is retarded only by the high prices of the iron works. Difficulties as to the drainage of this superstructure have not appeared; coarse, water-transmittent sand must be called the most appropriate material for ballasting. The superstructure with longitudinal bearers has especially given good service with sliding substructure. Our superstructure with wooden cross-ties has been considerably strengthened and has been made steadier by the introduction of the large tie-plates of my system.

You will see the details of these plates in the drawings enclosed. (Plates LIII to LV.)

Hoping that these communications will prove sufficient,

I remain,

Respectfully,

HOHENEGGER.

DISCUSSION.

WM. P. SHINN, Past-President Am. Soc. C. E.—Mr. President, I am glad to see somebody take up this subject of "permanent way" (as our friends across the water call it) and make some suggestions in regard to an improvement in the construction of our railroad tracks. It is now fully twenty-five years since I have dared to differ with most, in fact almost all of my co-laborers in the railroad field, in regard to one of the fundamental conditions of railroad track. Almost every engineer —almost every railroad man—will say, unquestionably and unalterably, the track must be "elastic." As has been suggested, we do not make the sub-structure that is to sustain all this weight elastic. We do not try to make the foundations of an immense building, so that it will yield when the weight is brought upon it, and the idea of there being a necessity for a railroad track to be elastic is, in my opinion, erroneous. As I said, I have held that opinion for twenty-five years. I have been on the other side of the fence. Our Member, Mr. Clarke, has got one foot over. He is proposing to make the track more rigid and give it a better support. I have taken the ground that a perfect track will be upon an absolutely rigid foundation, but it will also be absolutely a plane in its structure. What we need is a support which will hold the rails to the place where they are put. Mr. Clarke in his paper dwells upon the sinking of the joints, but he does not go far enough nor mention one-half the difficulty. Any one

who will watch the passage of one of our ordinarily heavy trains over even the best of our tracks, will see that every cross-tie, as the weight of the driving wheels and the loaded trucks come upon it, sinks down frequently half an inch and rises up again. What kind of a foundation is that, to carry at a speed of 50 miles or 60 miles an hour a machine which weighs 50 or 60 tons? There is another feature in this connection which seems to have been lost sight of. We know that the amount of power required to move our trains depends very largely upon the gradient up which they have to be moved, and the maximum load of our trains is determined by the maximum gradient over which the trains have to go, yet we are constantly providing a gradient of 1 to 2 per cent. in front of the driving wheels, so that even on a level road or foundation it is all the time climbing a gradient of 2 per cent. and sometimes 3 or 4 per cent. The experiments made by Mr. P. H. Dudley, C. E., with his dynagraph show that on the New York Central and Harlem River Railroad, a passenger train weighing 378 tons, moving at 55 miles per hour, required 720 horse-power on an 80-pound rail, and 820 horse-power on a 65-pound rail, an increase of 100 horse-power, or 14 per cent. Mr. Dudley expresses the opinion that on a very stiff 105-pound rail, the difference would be 200 horse-power, or 28 per cent., which goes to show that by giving the locomotive a firm support for the track upon which it lies, not only may we save immense sums in the maintenance of track, as has been pointed out in the paper, but we may very greatly increase the economical value of our power. Now, if that is true, the amount which such corporations as our trunk lines can afford to spend in order to get a perfect and rigid track is very large. It would probably justify an expenditure of two or three times the present expenditure on the track.

I hope at a future meeting of this society to submit some views on this subject which will be a little more definite than any which I can give you to-day. I hope to submit a more complete remedy for the problem than Mr. Clarke has given. His deductions are entirely in the right direction, but do not go far enough. If any of you have ever seen the freedom from resistance and shock with which a wheel moves over a plate of glass you can realize what would be the possible advantage of having for our trains a perfectly level and perfectly rigid track; so far we feel that we have been getting along with what was perhaps a necessity under certain economical conditions, but it is not a necessity any longer. The weight of our equipment, the volume of our traffic, the speed of our trains, all demand that we should have a more complete, a more perfect "permanent way."

Henry G. Prout, M. Am. Soc. C. E.—Probably no one will differ with what has been said as to the importance of increasing the stiffness of our railroad tracks. I merely rise to say that the track proposed by Mr. Clarke is apparently a very inadequate effort towards doing this.

You will see that the bearing surface between the longitudinal which carries one rail, and the ballast is only about 30 square feet; in ordinary track we have 48 square feet under one rail. With that inadequate bearing surface, as will be quite apparent to any railroad man, you would get serious rolling of your train, because it is hardly possible that the ballast should be equally firm on both sides of the track. There will be differences in the tamping or in the drainage sufficient to make one side deflect more than the other. Further, I do not see what there is to prevent the track creeping on grades. The creeping due to wave motion would not be so great as the creeping due to expansion, but the creeping due to expansion would be very great. The stringers would not be subjected to as great variations of temperature as the rails, but they would be subjected to great variations, and there is no reason why they should not expand down hill and contract down hill on grades. These, it seems to me, are fundamental and sufficient objections to this type of track.

James B. Francis, Past-President Am. Soc. C. E.—I am not a railroad engineer at all, but my recollections go back to the first railroads, those built in the old country and those built in this country. They were all laid upon stone blocks and, without knowing positively what the reasons were for the change from stone to wood, one thing that was said, I know, was that it pounded from non-elasticity in the track—that was what was prominently stated. They very quickly passed from stone blocks to the wooden sleeper, and the wooden sleeper has the elastic motion which the other had not. Not being a railroad man I cannot go into the reasons for these things; I merely speak of facts.

A Member.—I think the difficulty there, was that it was not a continuous bearing; undoubtedly it was a sort of bearing that would give very heavy shock. This was given as one reason why the stone blocks were taken out.

P. F. Brendlinger, Am. Soc. C. E.—The proof of the pudding is in the eating of it. I am very sorry that we have not more of the details of the Austrian system; we only have here an end view, cross-section and plan. It is not even shown how the ties connect the two longitudinal stringers except in the plan. The ties appear to be spaced 9 feet 9 inches and 10 feet apart. It is easy enough to build a railroad on a tangent and keep it in alignment. But it is quite a different thing with a curve, not only in regard to the elevation but also in regard to the alignment.

I believe that it will be a great many years before we will see this system, or anything similar, used in this country where there is competition and cutting of rates all the time (except in isolated cases) until you get considerable dividends on the investments. I know if this system were used on most roads there would be no dividends in a great many years. It seems to me that a stringer, which has such a sharp

knife edge, is liable to be depressed into the road-bed to such an extent whenever the train goes over it, that a very bad surface would prevail all the time. In summer this may be overcome by raising and tamping, but in winter the usual practice of "shimming" to keep a good surface will be difficult to perform. On embankments it will be difficult to keep a good surface, as the rains wash away the material of the road-bed down one side or the other. Fig. 1 shows a bent metallic cross-tie under a train. I never heard that that was the objection to metallic cross-ties. The great objection, as I understand it, is the tendency of the ties to move laterally. I have been informed that on the New York Division of the Pennsylvania Railroad, where metallic ties were tried thoroughly, various devices were used to prevent the side or lateral movement of the ties on curves without success. I got up a plan similar to this in Pittsburgh, in 1872, except that the stringers were made of wood. It was tried and then abandoned because it was much more expensive to keep up the track, and if the track was not kept in good order all the time there would be a seasick crowd of passengers on the train. I think it will be a great many years before this system is used in this country. We will not have it as long as we have so many competitive railroads.

O. Chanute, President Am. Soc. C. E.—I would say that it is my own recollection that at an early date a track system was tried in England, in which the rails were laid upon continuous walls in lieu of the stone blocks that were first used. Mr. George Stephenson tried once to make his road something like this, but it was said that those stone walls were found so rigid that they were abandoned. Then experiments were made with longitudinal systems, sometimes with the various sandwich girder systems in which the rails were bolted between two pieces of timber. There were quite a number of those shown in the report upon permanent way, published in 1858. That also shows the pillow saddleback rail, the bottom of which was made into two tongues which were fitted into the bolts. All these were abandoned for different reasons, chiefly, however, from the difficulty of side drainage, and it is possible these difficulties may be overcome by the proposed arrangement.

E. E. Russell Tratman.—Mr. Clarke refers briefly to the use of metal tie-plates between the rail and tie, and considers the joint to be the weak part of a track on cross ties. I think tie-plates are a very valuable means of improving the present tracks, until we are prepared to make a more decided improvement, and they are already being quite extensively used in this country and abroad. As to the joints, while there are various types and patents on the market and in use, I think a joint that will be really efficient under heavy traffic on a track laid with cross ties can be, but has not yet been designed. While I agree to some extent with Mr. Shinn as to the advantages of an absolutely rigid track,

based on the idea of smooth wheel rolling on a glass plate, I think the attainment of such a track very improbable, as it would involve much difficult work and heavy expenditure in general construction, apart from the track itself. The rail joint would be a difficult feature to manage, and even slight inequalities in track or rolling-stock, due to uneven wear, etc., would probably result in a rough riding road. A great increase over the present average degree of rigidity may, however, be practically attained by the construction of a more rigid track, particularly upon the already well consolidated roadbeds of the leading trunk lines. The rail joint question then again becomes important, in the endeavor to secure evenness of surface and consequent easy riding of the cars. The idea of the rigid railway is old. Mr. George Stephenson had one idea of fastening the rails down to solid rock, and he tried the plan in a rock cut on the Manchester and Leeds Railway, of which he was appointed engineer in 1839. According to published accounts the solid rock was dressed to a surface and the chairs were spiked directly to it, but the road was so rigid "that if a train passed over it at more than a walking pace, rails and springs were broken and in less than three weeks from the opening of the railway, orders were given for the rails to be taken up and placed upon sleepers in the ordinary way." The fastenings, however, probably worked loose, causing the battering of the rails between the wheels and the rock, and making a very rough road to travel over. If the rails could have been so securely and firmly fastened down as to become practically a part of the rock, which would have given absolute rigidity, it is possible that the results might have been somewhat different. Mr. Brunel originally designed the track for the Great Western Railway to consist of longitudinal timbers resting on transverse timbers placed upon piles, the piles being arranged in pairs at intervals of 15 feet. This was tried in 1838–39 between London and Maidenhead, but the ballast continually subsided, making a very elastic track with rigid supports at intervals of 15 feet. This track was soon taken up and its failure is generally attributed to its rigidity. Mr. Hartley built masonry walls upon which the rails were bolted, but "rails, tires and springs were broken daily and the plan proved an entire failure." On many of the old English railways stone blocks were used to support the chairs in which the rails rested, and it is stated that the riding over these blocks was harder than over wooden ties. This was probably due to the alternate elasticity and rigidity, and the perfectly natural result would be a more marked change from the suspended part of the rail to the rigid stone support, than to the semi-elastic wooden support. In some cases in India, where cast-iron bowl ties were laid in shallow ballast in rock cuts, it is said that the track was found to be too rigid, and it became necessary to put a greater depth of ballast under the ties. Mr. C. E. Stretton, an English engineer, makes the following remarks in his book on "Safe Railway Working:"

"Nothing can be worse than a rigid permanent way, but in the early days of railways this fact was not known or understood, consequently very many ideas and inventions proved failures. They provided a very strong road, but the rigidity was so great that the permanent way and rolling-stock were jarred to pieces, not worn out by ordinary working, thus clearly showing that a certain amount of elasticity was absolutely necessary. * * * Permanent way must be strong and firm, but at the same time possessing a certain amount of elasticity. It is very necessary that the elasticity should be uniform throughout, and not a system of alternate elasticity and rigidity, in which it serves to aggravate the rigidity by causing a succession of jumps and jacks." It may be, however, that if the rigidity was uniform throughout, the result would be even better.

In regard to experience with rigid road-bed and track on American railways, Mr. Charles Francis Adams has described a portion of the original track of the Boston and Lowell Railroad (opened on June 27th, 1835) as having the stone blocks, in which were inserted the oak plugs to which the rails were spiked, laid on a continuous foundation of parallel dry stone walls in trenches under each line of rails. These walls were $1\frac{1}{2}$ feet wide and from 2 to $4\frac{1}{2}$ feet deep. This construction did not prove successful, and it is stated that the directors "gradually learned to their surprise that speed without elasticity is always costly." A portion of the track of the old Columbus Railroad, of Pennsylvania, consisted of iron strap rails on continuous stone stringers 1 foot square, each line of stringers resting on a continuous bearing of broken stone filled into a trench.

James Owen, M. Am. Soc. C. E.—I noticed three or four weeks ago that the Great Western Railroad still adheres to the longitudinal stringers on their track; that was from London to Bristol.

C. E. Goad, M. Am. Soc. C. E.—That statement is true. Only three months ago I went over it. That portion of the Great Western passes mostly through an agricultural district. The gauge is still 7 feet, although there is a third rail for trains of ordinary gauge, and next year they will abolish the 7-foot gauge. The reason for keeping it is said to be that it is the only way they can compete with the South Western Railway to Exeter, which has a route 17 miles shorter. It always seems to me the longitudinal system of stringers has not had its fair trial, from the fact that a leading railway in England has for so long a time successfully maintained a long distance with this system.

E. P. North, M. Am. Soc. C. E.—Something was said about bolting a rail to solid rock. Possibly that was a mistake, as it would be almost impossible to find solid rock continuously. On the railroad from Albany to Schenectady, built by Mr. John B. Jervis about 1831, the rails were laid on longitudinal timbers which were fastened to blocks of limestone about 2 x $2\frac{1}{2}$ feet in place and rather over 1 foot thick. The longitudi-

nals were fastened to the limestone blocks by cast-iron angles; these were spiked into wooden trenails in holes drilled for them and to the sides of the timbers. I think the whole distance from Albany to Schenectady was laid with these blocks, and a part of the road from Schenectady to Saratoga. These were all taken out about 1845, as the timbers were battered to pieces under the locomotives, which did not weigh more than 5 tons. They were replaced by pieces of plank 3 inches thick and about 3 feet long. These blocks, I think, were used in Pennsylvania with the same result.

Mr. TRATMAN.—In regard to Mr. Stephenson's experiment, it is only a matter of memory, but I think it was tried as an experiment only in the rock cutting near Liverpool.

P. F. BRENDLINGER, M. Am. Soc. C. E.—Mr. Shinn spoke about having a good rigid roadway. I have wondered how we are going to get it. We would have to change our method of building railroads. If you take out a cut what will you do with the material? It is put in embankments. There are plenty of embankments 50 or 60 feet high, often more. We always allow in construction a certain percentage for shrinkage, and the banks keep on shrinking for years, as every superintendent of a railroad knows, and he has to keep raising the track all the time. How can there be a rigid substructure in a case of that kind?

HUNTER MCDONALD, M. Am. Soc. C. E.—It strikes me that in some instances that difficulty may be overcome with piles. Piles shod with iron could be driven through a loose rock embankment to the original surface. There is one point that has probably escaped the attention of members, and that is the advantage that cross-ties offer in case of derailment. Trucks sometimes leave the rails and run on the ties for miles without accident, but should this occur where the track was laid on longitudinal stringers, such derailment would be likely to overturn the train.

J. FOSTER CROWELL, M. Am. Soc. C. E.—I regard this paper of Mr. Clarke as both valuable and timely. Owing to the causes which the author has so tersely and comprehensively enumerated, the era is at hand when railway companies will be brought face to face with the absolute necessity for providing tracks of adequate strength and endurance to withstand the exigencies not only of the present conditions of traffic, which have, in many cases, almost reached the proper limitation of the modern track structure, but of the inevitable future increase in loading and speed which economy in operating the railroads demands. The civil engineer cannot stop the process of evolution that applies to traffic movement, and must meet and anticipate its requirements for satisfactory structures. It is quite apparent, and could be demonstrated by a simple presentation of the statistics of the operation of railways with large traffic, that the excessive cost of maintenance of way represents a very large sum in the aggregate which, if capitalized, would justify a much greater

outlay for track than railroad managers venture to expend in first cost; and the conjectural advantages of additional safety, fewer wrecks and less wear and destruction of rolling stock can be estimated at perhaps almost an equal figure.

Moreover, if a track can be constructed of such perfection that the heaviest trains can be of maximum loading, moved over it at twice the present attainable speed for that class of trains, the sum required to build an additional track of the present type could be added to the improvement account without transgressing the laws of true economy. I believe that a longitudinal bearing system, in some form or other, is the most promising solution of the problem, but am not prepared to agree that independent bearers, connected flexibly, will afford the full measure of security and permanency required in a typically perfect track. Continuously longitudinal support for the rails is essential, but there should be in addition a rigid supporting lateral system on most road-beds in all exposed situations and especially on curves; and first of all the road-bed itself should be of an enduring and unchangeable type, not affected by water, frost or wear. The present road-bed, at its best, is an anomaly; what would be thought of the wisdom of erecting a permanent building upon a foundation requiring the unceasing efforts of a standing army of workmen to make it serve its purpose? And yet this is what has to be done, yesterday, to-day and forever on every railroad in this country; and despite this vast expenditure of costly effort, each advance in volume and weight of traffic only brings the structure nearer to the point of failure. I do not think that any system of track structure that can be devised can be rendered effectual and economical on present road-beds, and that any new system should include the substructure.

T. C. Clarke, M. Am. Soc. C. E.—In replying to the discussion, I would say I am much gratified at the interest shown by members of the society in the important subject of my paper, and their belief that there is room for improvement of our tracks. Mr. H. G. Prout is mistaken in supposing that I recommend the Hohenegger system, or any specific system. My intention in this paper was to indicate certain general lines of investigation. The point referred to by Mr. Prout, of insufficiency of bearing surface, is one of such great importance, and its evil effects, as shown by Mr. Shinn, from the experiments of Dr. Dudley, are so conspicuous that I will here go much into detail. My observations show that an 80 pound rail will distribute the weight of one 8-ton wheel over two ties placed 2 feet apart; thus each tie carries 4 tons. This load does not sink the ties into the ballast over one-eighth of an inch, which corresponds to a grade of 13 feet per mile. These ties have 8 inches face, by a length available for bearings on each side of 2 feet, or 4 feet in all, giving a bearing surface of 2.64 square feet. The ties being 2 feet apart between centers, we have a bearing area per lineal foot of rail of 1.32 square feet. As some wheels carry more than 8 tons I have

assumed 1½ square feet as the least allowable bearing surface per lineal foot of rail of a stiffness equal to 80 pounds section. This is equal to a pressure of about 1½ tons per square foot of bearing surface with the wheel loads now in use.

I know of but three modes of supporting railway rails: by cross-ties, by the pot system and by longitudinal bearers. It may be interesting to compare the dimensions and weights of tracks on these three systems, worked out on the basis of allowing a bearing surface of 1½ square feet per lineal foot of rail.

These systems are all metallic, and to give as good a grip upon the ballast as the standard wooden cross-tie system has, the cross-ties, pots, and longitudinals must have flanges running down at least 6 inches into the ballast. The cross-tie system has ties 9 feet long by 12 inches bearing surface, which are placed 2 feet 8 inches between centers. These ties would average 300 pounds each, if of metal one-half inch thick; and the weight of metal per yard would be:

	Pounds.
Per yard	333
Two 80-pound rails	160
Per linear yard	493

The pot system would have stamped plates 2 feet 2 inches square by one-half inch thick placed 3 feet apart between centers, united by light cross-ties, weight—

	Pounds.
Two pots, 120 pounds	240
Tie	10
Two 80-pound rails	160
Per linear yard	410

The longitudinal bearers would be 18 inches wide by one-half inch thick, united by light cross-ties:

	Pounds.
Two bearers, 120 pounds	240
Tie	15
Two rails	160
Per linear yard	415

This track would have weight enough, bearing enough, and grip enough to lie still and not deflect under passing loads more than enough to give a 13-foot grade per mile.

Many inventors are striving to produce a cheap metallic cross-tie system with ties weighing under 150 pounds each of 6-inches face, 7 feet long and placed 3 feet apart. Such a track would weigh—

	Pounds.
Ties	150
Rails, two of 80 pounds	160
	310

per linear yard, or much less than any as above described. How is this economy obtained? By cutting down the bearing surface to one-half a square foot per linear foot of rail, or one-third of what it ought to be, thus increasing the subsidence of the ties into the ballast and increasing the grade of the hollow, out of which the locomotive is always striving to climb, to 40 feet per mile. To avoid this, rails would be required of a section weighing 120 pounds per yard, and thus would take away the supposed economy.

Referring to the views of Mr. Shinn, who thinks that the foundations of rails should be rigid and unyielding, it seems only necessary to point out that wheels cannot be maintained truly circular. Therefore shocks and blows must take place between wheel and rail. If the latter is unyielding it would be battered down and the wheels also would suffer. But by a little yielding, not over one-eighth of an inch per tie, or about 13 feet per mile, they would cushion these blows entirely.

Finally, tracks made heavy enough to give the necessary bearing of 1½ square feet per lineal foot of rail would cost double as much as our present wooden cross-tie system does. But the locomotive would haul more cars and at a higher speed, and owing to its greater strength and the absence of wooden material, the saving of cost of repairs would more than recoup the interests on the increased cost, which at five shillings would be $385 per mile per year.

ERRATA.

Page 136, second line from bottom, Plate VII should read Plate XVIII.

Page 235, fourth line from bottom, Plate LII should read Plate XLII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

497.

(Vol. XXV.—September, 1891.)

YIELD OF THE SUDBURY RIVER WATER-SHED IN THE FRESHET OF FEB. 10TH–13TH, 1886.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

One of the most disastrous rainstorms that has visited New England since accurate records have been kept, occurred in February, 1886. About 8 inches of rain fell over the greater part of the State of Rhode Island, which included the center of the storm. In Massachusetts the rain was less severe, although even in this State a great deal of damage was done. It is obvious that had the heaviest part of the precipitation fallen on the upper portions of the rivers instead of near the sea, the effects would have been far different.

Professor Winslow Upton, of Providence, estimated the areas covered by different amounts of rainfall as follows:

AMOUNT OF RAINFALL.	AREA IN SQUARE MILES.
Over 8 inches.	750
Between 7 and 8 inches,	750
" 6 " 7 "	1 500
" 5 " 6 "	1 850
" 4 " 5 "	2 750
Total above 4 inches,	7 600

A considerable amount of snow and ice covered the ground at the beginning of the rain, averaging, according to general testimony, about 2 inches of melted snow. Some of the rain as it first fell was absorbed by the snow. The temperature during this portion of the storm was near the freezing point, so that the snow melted slowly, but perhaps three-quarters of it went off with the rain, and the remaining portion followed a few days after the end of the rain. This kept up the flow of the streams in a remarkable manner long after the rain had ceased, and was one of the characteristics of the freshet. The ground was frozen, and when it is in this condition, the frost acts like a waterproof cover over the surface.

The rain in the Sudbury River water-shed began at 7 P.M. of the 10th, and ended at midnight of the 13th, thus extending over three days. The amount of rainfall was 4.64 inches, which with the melted snow made 6.64 inches total precipitation. The greater part of the rain fell during the afternoon and night of the 12th. The rise of temperature, which aided in carrying off the snow, is shown by the daily maxima of temperature observed. On the 11th and 12th the thermometer registered 37 degrees Fahr., but rose to 46 degrees on the 13th, 51 degrees on the 14th, and 54 degrees on the 15th and 16th. On the morning of the 16th the snow had all disappeared with the exception of a little in the woods.

The writer has been unable to find a continuous record of the rain on the Sudbury water-shed, but from notes and comparison with records in other places, he is led to believe that the curve of rainfall as assumed and drawn on the accompanying diagram is quite near to the true distribution of the rain. At Hopkinton 4.76 inches, and at Lake Cochituate 4.95 inches, were reported. The former is within the area which we are discussing and the latter contiguous to it. These observations confirm the accuracy of the total rainfall, as determined by the rain gauge at Framingham.

It is unnecessary to discuss the value of accurate river gaugings in time of freshet flow. The hydraulic engineer has frequent occasion to use such data in the course of his practice. It is to be regretted that more information of this kind has not been published.

The Sudbury River water-shed is one of the sources of water supply of the City of Boston. It is 25 miles west of the city. Its area is about 75 square miles, varying slightly from this amount, as is shown

Yields of Sudbury River and Cold Spring Brook Water-sheds during the Freshet of February, 1886.

	Sudbury River Water-shed. Area = 74.656 square miles.....13th to 18th. " 75.199 " " .... {6th to 12th inclusive. 19th to 24th "}				Cold Spring Brook Water-shed. Area = 6.434 square miles.				
	Yield in gallons.	Yield in gallons per sq. mile.	Inches in depth on water-shed.	Points for cumulative Profile.	Yield in gallons.	Yield in gallons per sq. mile.	Inches in depth on water-shed.	Points for cumulative Profile.	Remarks.
February 6..	79 140 000	1 052 000	0.061	—0.287	6 300 000	979 000	0.056	—0.2615	
" 7..	78 240 000	1 040 000	0.060	—0.226	5 880 000	914 000	0.053	—0.205	On February 13th the yield of the Sudbury River water-shed was as follows:
" 8..	81 090 000	1 078 000	0.062	—0.166	5 880 000	914 000	0.053	—0.152	
" 9..	92 220 000	1 226 000	0.071	— .104	7 270 000	1 130 000	0.065	—0.099	
" 10..	86 180 000	1 146 000	0.066	{— .033 * 0.000 0.033}	7 720 000	1 200 000	0.069	—0.0345	Seven A.M. to noon, at rate of 2 136 million gallons in 24 hours. Noon to 4 P.M., at 2 060 millions in 24 hours. 4 P.M. to 7 P.M., at 2 087 millions in 24 hours. 10 P.M. to midnight, at rate of 1 997 millions in 24 hours. Midnight to 6 A.M. on the 14th, at rate of 1 647 millions in 24 hours.
" 11..	170 740 000	2 270 000	0.131	0.164	20 980 000	3 261 000	0.188	{0.0000 * 0.0345 0.222}	
" 12..	943 210 000	12 543 000	0.722	0.886	106 140 000	16 496 000	0.949	1.171	
" 13..	1 997 550 000	26 757 000	1.540	2.426	201 380 000	31 299 000	1.801	2.972	
" 14..	1 289 600 000	17 274 900	0.994	3.420	109 240 000	16 979 000	0.977	3.949	
" 15..	838 210 000	11 228 000	0.646	4.066	69 090 000	10 738 000	0.618	4.567	
" 16..	531 120 000	7 114 000	0.409	4.475	46 050 000	7 157 000	0.412	4.979	Rainfall—7 P.M., Feb. 10th, to midnight, Feb. 13th, 1886.. 4.64 inches.
" 17..	428 510 000	5 740 000	0.330	4.805	27 020 000	4 200 000	0.242	5.221	Snow on ground..... 2.00 "
" 18..	357 190 000	4 784 000	0.275	5.080†	23 390 000	3 635 000	0.209	5.430‡	
" 19..	395 550 000	5 260 000	0.303	5.383	32 920 000	5 117 000	0.294	5.724	Total rain and snow.. 6.64 "
" 20..	336 430 000	4 474 000	0.257	5 640	24 450 000	3 800 000	0.219	5.943	
" 21..	269 270 000	3 581 000	0.206	5.846	24 410 000	3 794 000	0.218	6.161	Shower of Feb. 15th, 8.30 P.M., 0.02
" 22..	265 020 000	3 524 000	0.203	6.049	15 750 000	2 448 000	0.141	6.302	
" 23..	168 120 000	2 236 000	0.129	6.178	15 050 000	2 339 000	0.135	6.437	Feb. 19th, 4 P.M. to midnight, 0.66
" 24..	172 720 000	2 297 000	0.132	6.310	11 820 000	1 837 000	0.106	6.5435	

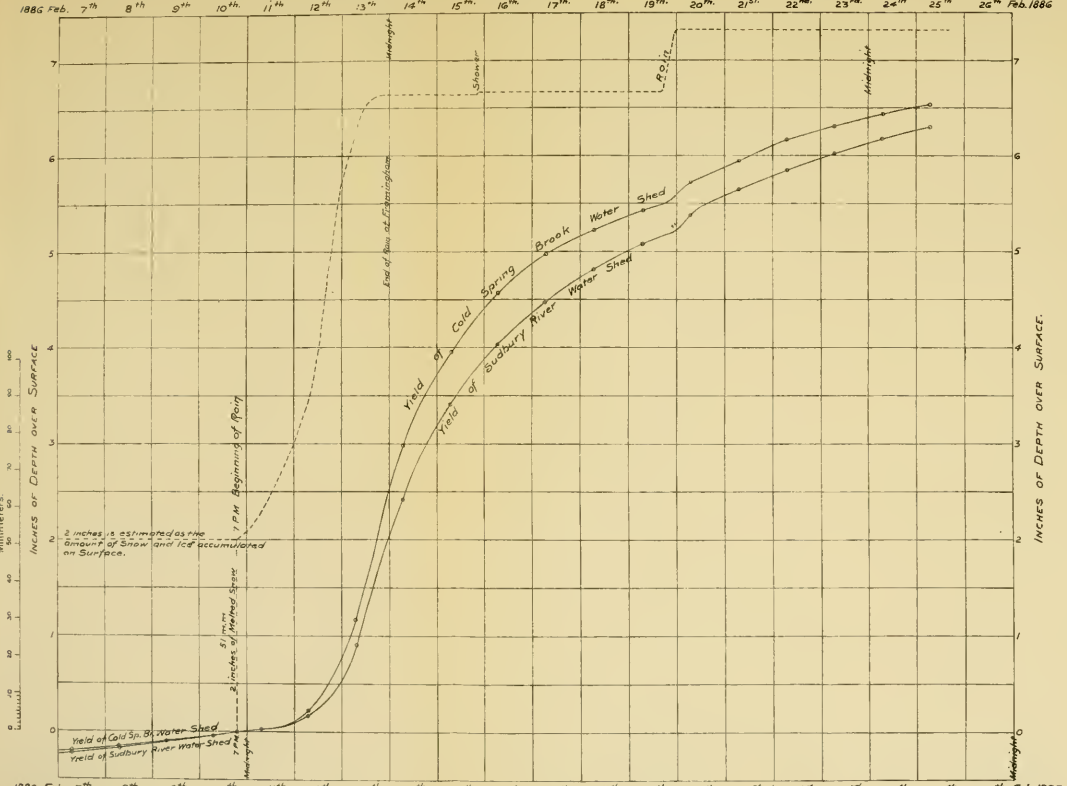
The Sudbury River water-shed includes that of Cold Spring Brook.

*Beginning of rain = 0.000. †5.08 inches collected. ‡5.43 inches collected. 6.66 total rain and snow.

PLATE LVI
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXV. No 497.
FITZGERALD ON
YIELD OF THE SUDBURY RIVER

FRESHET OF FEBRUARY 10TH–13TH 1886.

Cumulative Diagram of Simultaneous Precipitation and Yield of Sudbury River and Cold Spring Brook Water Sheds.



in the accompanying table. The topography represents a fair average New England water-shed. About 38 per cent. of the surface is covered with woods. The Cold Spring brook water-shed forms a fraction of the above area. It covers 6.434 square miles. The boundaries of both water-sheds have been determined by survey. The gaugings are made daily at 7 A.M. The yields are determined by the flows over the dams at the outlets of the respective water-sheds, modified by the gains or losses in the storage basins upon their surfaces. The accompanying table gives the yields in gallons; the yields per square mile in gallons; these yields reduced to their equivalent depth in inches upon the water sheds; and in the final column in each case, the successive additions from the time when the rain began, the latter point being taken as zero. The yields preceding this time are tabulated as minus quantities in order to separate them from the total collections which follow and which are due to the rain.

The writer has plotted three lines upon the diagram which accompanies this paper. The upper line is the rainfall. It has been started 2 inches higher than the lines which are about to be described, to allow for the 2 inches of snow upon the ground at the beginning of the rain. The other two lines represent the yields of the two water-sheds in inches in depth, and starting from zero at the beginning of the rain they represent at any point the total yield at that point reckoned from the beginning of the rain. It will be noticed that the yields previous and subsequent to the rain were small, and they are shown in the diagram by the flat inclination of the lines of flow as they approach the storm; consequently the large flows were due entirely to the particular storm we have been discussing. For this reason it is safe to assume that out of the 6.66 inches total rain and melted snow came the 5.08 inches on the Sudbury, and 5.43 inches on the Cold Spring brook water-sheds which were collected between the beginning of the rain and the morning of the 19th, five days after the rain ceased.

It will be noticed that the maximum yield per 24 hours on the Sudbury water-shed was equal to 1.54 inches in depth upon its surface, and on Cold Spring brook water-shed to 1.801 inches. The maximum rate of yield on the larger water-shed occurred between 7 A.M. and noon of the 13th. For this period the flow averaged at the rate of 2 136 000 000 gallons in twenty-four hours, or 1.646 inches in depth on the water-shed per day. On March 26th, 1876, on the Sudbury River water-shed, almost

exactly the same yield was gauged, viz., 1.559 inches in depth per twenty-four hours on the water-shed.

The dams on the Boston Water Works are proportioned for a continuous flow of 6 inches depth per day over their water-sheds. This is not an unreasonable provision for maximum flow, as will be apparent on careful consideration. Mr. James B. Francis, M. Am. Soc. C. E., has given this Society the results of his investigation of the great storm of October, 1869,* when 12.35 inches fell at the center of the storm, in Connecticut. At Middleton, 7.15 inches fell in twenty-four hours, and it is possible that at the center of the storm considerably more than 8 inches fell in one day. If such a rain as this should fall on frozen ground covered with a body of snow, and accompanied by a high temperature, it might create a flow equal to 5 or 6 inches in depth upon a water-shed in twenty-four hours.

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* *Transactions*, Vol. 7, page 224. August, 1878.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

498.

(Vol. XXV.– September, 1891.)

NOTES ON CEMENTS, MORTARS AND CONCRETES.

By WILLIAM H. GRANT, M. Am. Soc. C. E.

WITH DISCUSSION.

The rules formulated by the Committee of the Society on a uniform system for tests of cement, with notes as to the quality and treatment of the material, etc., have been of great value, as shown by their general adoption by engineers throughout the country. They were a step forward when much needed; and the author thinks it would not be too much to say that no greater service has been rendered to the profession by our Society than by the bringing out of that paper. While such commendable service has been rendered by the labors of the Committee, many other papers on the same general topic have been contributed by other members to the *Transactions* of the Society; these, with the discussions they have elicited, would seem to have so far exhausted the subject as to make it a venturesome business to approach it with new matter. There is still, however, room for expansion—room for additional information, and rules for guidance in practically dealing with this invaluable material in the many details of everyday practice. Much as we know about its remarkable properties, it must be admitted that there is yet a good deal that we do not know.

It has been said that "mortar is the soul of masonry," and masonry constitutes not only the soul but the body of most engineering constructions. The author does not set himself to the task of evolving anything especially new upon the subject. This remains for the few in the profession who have the time, means and other requisites for the object, and who may have the laudable desire to pursue the search to its fuller development. He merely aims in this paper to call attention to such needs and deficiencies as pertain rather to our present knowledge and practice, and to make a few suggestions in regard to them, at the same time trespassing as little as possible on previously well-trodden ground.

Reference is now made to notes taken while superintending the construction of the new Naval Observatory on Georgetown Heights, Washington, D. C. This work consisted of a group of nine buildings, a sketch of the positions of which, on a small scale, is appended hereto. They are designated as per sketch (Plate LVII):

No. 1 Main building, 69 x 307 feet; No. 2 Grand equatorial, 46 x 72 feet; No. 3 Clock-room, 18 x 20 feet; Nos. 4 and 5 Observers' rooms, 18 x 20 feet; Nos. 6 and 7 Transit circle buildings, 30 x 40 feet; No. 8 Prime vertical building, 18 x 20 feet; No. 9 Boiler-house, 45 x 54 feet. *A*, *B* and *C* are underground ducts for steam heating and water-pipes, connected with the boiler-house.

They are all fire-proof, except Nos. 4 and 5. These, in consequence of the stinted appropriation, had to be economized in superstructure, although the foundations and basements were constructed in conformity with the other buildings. It is expected that the cheaper work will be replaced later with marble and iron. Nos. 1, 2 and 3 are of marble, with foundations of gneiss, the backing and interior walls of brick. Nos. 6 and 7, foundations and basements of gneiss, with iron superstructures. A third building of the same kind forms a part of the main building at the west end. No. 8 is the same as Nos. 6 and 7. No. 9 has exterior walls of broken range gneiss, with interior walls of brick; it is situated on side-hill ground, and has a flat roof of iron beams and brick arches, with paved surface for driving on with coal carts. The foundations are deep and walls heavy. The work was done under contract with the Navy Department. The specifications were prepared by the architect, Mr. R. M. Hunt, and were full and stringent. It is to be observed that the contract was drawn so as to allow but little discretion or latitude in the interpretation of its spirit and letter.

The restrictions were rather in excess of those in contracts for work other than for the United States Government. Officers and agents were held to a strict accountability, and it required great care not to overlook stipulated conditions or to exceed them in any respect; such precautions having doubtless been induced by the necessity of avoiding the giving to contractors a pretext for extra claims, which if permitted might make serious inroads upon the United States Treasury.

Competition was spirited in bidding for the work, and it was let for a lump sum to contractors on terms which afforded but a small margin for profits. The most of the materials had to be hauled from 1 to 3 miles, from city wharves, brick kilns, etc., and to surmount on the route an elevation of about 300 feet. Two years were allowed for the completion of the contract.

The cement, brick and marble brought to the work were variable in quality—the marble in both quality and workmanship; in round numbers, about 20 per cent. of each had to be rejected. It may be said here, as one of the points to which attention will be called later, that the rejected cement and brick as removed from the work were understood generally to have been disposed of and used in construction in the City of Washington. Marble which was rejected for defective workmanship only was made good as far as practicable, but being dimension stone it was not easy to remedy the defects. The requirements for cement were, "best imported Portland," and the tests established at the commencement of the work were:

For fineness—To pass a sieve of 2 500 meshes, with a residuum of not over 5 per cent.

For set—Neat, the initial set to be in not less than twenty minutes.

For tensile strength—For seven days, 360 to 375 pounds.

The tests for domestic cement were:

For fineness—To pass a sieve of 2 500 meshes, with a residuum of not over 8 per cent.

For set—Neat, initial set in not less than twenty minutes.

For tensile strength—Neat, one day, 40 to 50 pounds; seven days, 90 pounds.

It will be seen that slow-setting cements were wanted.

These tests were not rigidly adhered to. The impossibility (as claimed by the contractors) of procuring cement in quantities neces-

sary for carrying on the work uninterruptedly, that would bear the tests, induced some modifications of them; as, for instance, if the degree of fineness was not quite in accordance with the requirement, and tests of the same sample showed a satisfactory set and an excess of strength, the cement was accepted. Later on, as the works neared completion, a further modification of tests for the domestic cement was made, as will be again referred to. The domestic brands were mainly used.

The specification for brick was for "best common hard-burnt brick, of good shape," a sample of which was presented at the Navy Department prior to the letting of the contract.

It may be inferred from this brief description of the work, specifications, contract and materials to be dealt with, that some considerable care and responsibility devolved upon the Superintendent. It is one of the disagreeable duties of an engineer to look after defects and failures as well as successes; there is professional profit in it, if no other consolation. It is proposed to enumerate some of the difficulties met with. First and foremost, and throughout the entire course of construction, was the cement difficulty. The cement first used was the Portland, in the concrete foundations. Six different brands were tried; they are among the best brands in ordinary use. After careful testing only two were retained. Preliminary to testing, it was desirable to know the age of the cement. This, in no case, could be ascertained; when shipped, how long it had lain in warehouses, and what exposure it had encountered, were all unsolved problems. The general reputation was ascertained as far as practicable before ordering it, by referring to other works where it had been used. Then followed the tests. It was expected that full weight of barrels and a reasonable uniformity of quality would be obtained. It was found that the numerous barrels weighed, of three different brands, ranged from 368 to 402 pounds net, making differences of 34 pounds; the average weight was 378 pounds per barrel, the deficiency being 18 per cent. under the standard weight of 400 pounds net. This irregularity was not so troublesome as the variations in quality; deficient barrels could be replenished when they were liable to cause material differences in the proportions due to batches of mortar or concrete. The loss of weight fell upon the consumer. The fineness of this cement was quite even, and mostly within the requirements of 5 per cent. The initial set was extremely variable; the extremes were from five to one hundred and twenty minutes; the general range was from seven-

teen to sixty minutes. The set with heavy wire was from sixty to two hundred and forty minutes; the initial set with sand, 2 to 1, was two hours and forty minutes to three hours and fifty-one minutes. The general difference in initial sets, between neat and 2 to 1 was about two hours.

In tensile strength it followed a similar irregularity. One-day tests, neat, gave from 50 to 200 pounds; general average, 108 pounds. Seven-day tests (the lowest of a few) 86 pounds. The general range of lowest, 322 pounds. Average of all, 369 pounds; the highest of a very few, 700 pounds. Two to one tests for seven days gave 150 to 280 pounds, with an average of 188 pounds. A few twenty-eight-day tests gave 405 to 670 pounds. Water (applied, of course, in accordance with rules laid down by the Committee of the Society) varied from 19 to 30 per cent.; the general range was from 25 to 30 per cent. Of the two best brands used one took much more water than the other. Of course, with such a record, a good deal of the cement had to be rejected. The rejections were principally for too quick setting. A large amount of labor was required in testing. As it was impracticable to fully test every barrel, some were taken (except in a special case to be mentioned) with only a test for a set.

Coming to the domestic cement, of which much larger quantities were used, the difficulties were still further enhanced. Eight different brands were obtained, but as they fell out of the ranks from time to time on being tested, only one brand was continued in general use. This was a cement which is very largely used in the City of Washington in both public and private buildings. It is from the upper Potomac, and when good, is exceedingly good; there is but little choice between it and Portland for ordinary work. It is unfortunately very variable, fluctuating between the best desirable quality and a quick-setting or weak material. The fault can only be attributed to carelessness and indifference in its manufacture. It would seem that there is no spur to better efforts, from the remarkable fact that much of it is used, taken upon trust by consumers without testing, or with inadequate or unreliable tests. This refers to a previous remark as to the use of cement which was rejected at the Observatory work.

Among other brands that were rejected as falling short of the moderate requirements made was a New York brand, one that has borne a good reputation and has been extensively used, but an entire cargo of it,

shipped for the Observatory work, had to be rejected. It was thoroughly tested, as the importance of the case required, but it would not meet the requirement, barely reaching a tensile strength of 20 pounds on a one-day test. Thinking that the transportation by water might have had some effect upon it, the author had a sample expressed by rail from New York; it was received in apparently good condition and was tested, but it showed no improvement. Again as much of it was coarsely ground, a sample was screened out, using only the finest material, with the same result. A peculiarity of the cement was a sprinkling throughout it of fine particles of unconsumed coal. It set neat in eighteen to seventy minutes, mostly in thirty to seventy minutes; the tensile strength, one day, was 15 to 25 pounds, average 20 pounds; the strength for seven days was 26 to 100 pounds, average 51 pounds. Water used in briquettes 33 per cent., with but little variation.

The Navy Department was so desirous that no injustice should be done in the matter, that an expert was employed from the Navy Yard who devoted a good deal of time in making tests wholly independent of mine, and his tests confirmed the tests I had made. The author wrote to the agent of the cement company in New York to inquire if he could not supply cement with a guarantee that it would stand the moderate tests required. His reply was that he would cheerfully do so, if the tests were made in New York. As this was an impracticable condition no more came of it. The history may be completed by adding that the author was informed at the office of the Inspector of cement and asphalt for the District of Columbia, that the best quality of Rosendale cement was seldom sent to the Washington market. It will also be not out of place to say that the District Commissioners prescribe a lower test for Potomac cement than for Rosendale.

To return to the Potomac cement; several thousand barrels were used; the consumption at the greatest was about fifty barrels per day, and the perplexities attending its use were great. In general there was a scanty supply, the contractors were wary about laying in a large stock; and as this precaution was not diminished as the testing progressed, the consequence was that the work had to be slowed up, and sometimes brought to a standstill, to await the delivery of further supplies and give time for testing. The necessity for unwearied and uninterrupted tests will be seen from the results that were obtained. The requirements for the cement have been stated; as a rule it was used in mortar

of two parts of sand to one part of cement, and a quality of mortar was wanted that could be used (except in some cases of slow-setting Portland) in half an hour from the time of mixing, and thus avoid the liability of a set intervening with the necessity of remixing and the attendant deterioration.

The average weight of barrels was 280 pounds net, varying from 256 to 334 pounds, making an extreme difference of 54 pounds. The net weight as a whole was 7 per cent. below the standard of 300 pounds. A small portion was furnished in bags, the weight of which held out in full, as they generally weighed 150 pounds—two bags being the equivalent of a full barrel. Samples were tested that would show an initial set of three to ten minutes, sometimes with a moderate tensile strength, and then again wholly worthless; entire lots had to be rejected. A delivery of two or three hundred barrels would occasionally run quite uniformly, of excellent quality, and then be followed by the previous kind. It was remarkable how suddenly these transitions would occur. It required much watchfulness to detect the turning point. The manufacturers, when this state of things was pressed upon them, would make an effort to supply better material; it would succeed temporarily and soon fall off again; mixed supplies would reappear and these would fail in quantity as well as quality. Before the culling process of testing could be completed exhaustion would be near at hand, and delays of work would ensue before another arrival, or before it could be tested, etc.

The alternative was often unpleasantly presented of straining a point to help the contractors along, or to incur their complaints of a too stringent enforcement of rules. Quantities of rejected cement would accumulate and it looked to them like a hardship to be prevented from using it and to be obliged to incur the expense of hauling it away. As one difficulty was with quick-setting cement it was permitted to be used, with the restriction of using the mortar in twenty minutes after mixing, but it was not easy to obtain a compliance with such requirements. An expedient was tried of mixing 20 per cent. of lime paste with the quick-setting cement to prolong the set. It proved so unsatisfactory that it had to be discontinued; it increased the set but diminished the strength and it was impracticable, with ordinary vigilance, to obtain a proper intermixture of the materials; streaks and blotches of lime would appear in the resulting mortar, which evidently impaired its quality. All the

mortar and concrete used in the work were hand-mixed. It would have been much better if machine-mixing could have been substituted.

The author is glad to be able to bear testimony to the excellent qualities of this cement, which came like angels' visits now and then to the work. The general range of such cement was, for tensile strength, neat, for seven days, 100 to 191 pounds; several samples went up to 200 and 250 pounds; the highest of a few twenty-eight-day tests was 270 pounds. This shows that the manufacturers can produce an excellent article if they make the effort.

So much embarrassment was caused in the progress of the work that, finally, as the walls of the several buildings had reached near the point for topping them out, the cement requirement was reduced. This was in consequence of the expressed desire of the Navy Department to relieve the contractors to the fullest extent practicable in the difficulties they claimed to be laboring under, of procuring cement in quantity and quality called for. It was also important to expedite the work, as the term originally specified for its completion had expired. The author was informed by the Department that the Government engineers on other works were accepting the cement on a test of 30 pounds tensile strength for one day. His tests were accordingly reduced thereafter to that standard. A set of ten minutes had previously been accepted with the conditions mentioned.

Such were the difficulties and perplexities that were encountered in dealing with the cement question. How much of the same kind have been met with by other engineers the author is unable to say, but he cannot suppose that his experience has been exceptional. He has recounted them in order to call attention to facts which are of importance and which seem to be deserving of some method of alleviation. He is aware that there are some builders who are prone to say of their work that it is "good enough," and that a persistent adherence to troublesome rules of construction is "more nice than wise," but the adage that "whatever is worth doing at all is worth doing well," is an old and safe one, and in this age of scientific progress it is scarcely excusable for an engineer to content himself with thumb rule practice for which he cannot render a reason and demonstrate that it is the best the case will admit of; his work must be either good in the fullest sense of the word, or indifferent, and but few, he imagines, are willing that it should be placed in the latter category.

Some points may now be noticed which bear upon the general subject, and which, though perhaps not new, will admit of repetition. These relate to practices in connection with the use of cements and mortars which are draw-backs more or less upon successful work.

First.—Mixing Mortar.—The advantages of machine mixing are well known. In hand-mixing the first draw-back is the liability of errors in measuring out correct and uniform proportions of prescribed materials. Mortar men make mistakes which generally happen to be against the proper proportions of cement; the quantity of sand will also vary according to whether it is measured in a wet or dry condition, packed or loose. Next, they fail to intermix the cement and sand thoroughly before adding the water—an important point. Again, they will ease up on the labor required to mix all well together after applying water, and to facilitate the operation will over-dose the water. A further error occurs in assuming that all barrels of cement contain equal quantities. The necessity of a close supervision will be recognized in these particulars.

Second.—Second Set.—The mortar is placed on the work with the intention of its being used within a specified time before it has taken its initial set. Masons like it extremely plastic, and before their mortar board is emptied they will make frequent calls to "temper up;" more water will be added with re-mixing, and, if oversight is relaxed the prescribed time of using it will have elapsed and a diluted, weakened and second set material will have been used. The second set is so desirable, and its harmlessness so believed in by masons, that they will obtain it if practicable. Brick can be laid with more ease and rapidity with such mortar than when it is in proper condition. The masons claimed that it was permitted on other work, and that it was an unreasonable restriction to prevent it; that they could do more work and with more ease by the use of it. The author was unable from personal observation of the practices on other works to refute their premises.

Third.—Laying Brick without Wetting.—This practice when indulged in is the cause, as much observation and many results have shown, of a large amount of bad work; it is too patent to be remarked upon. Men do not like to handle wet brick; their mortar does not work so well with them; is liable to "run," etc., and it is disagreeable to them. A similar trouble occurs with stone work. Stones heated in the sun need cooling

off with water before laying them in a bed of mortar, and wet stones are objected to as unpleasant to handle and wearing upon the hands.

Fourth.—Bedding Brick.—The specifications for this in the Observatory work were, to "lay in full bed of mortar, filling end and side joints in one operation, and well bedded in place so close that nine courses of brick laid wet measure 2¼ inches more than nine courses laid dry." This was equivalent to requiring ¼-inch joints. The "bedding at one operation" is simple and easy with skillful masons—if they will do it—but it needed a good deal of persistence to get it accomplished. "Full bed and end joints" was another difficulty hard to overcome—¼-inch joints also. Masons have a habit of laying brick in a bed of mortar leaving the vertical joints to take care of themselves, throwing a little mortar over the top beds and giving a sweep with the trowel which more or less disguises the open joints below. They also have a way, after mortar has been sufficiently applied to the top bed of brick, of drawing the point of their trowel through it, making an open channel with only a sharp ridge of mortar on each side (and generally throwing some of it overboard), so that, if the succeeding brick is taken up, it will show a clear hollow, free from mortar through the bed. This enables them to bed the next brick with more facility and avoid pressure upon it to obtain the requisite thickness of joints. With ordinary interior work a common practice is to lay brick with ½ and ¾-inch mortar joints; an inspector whose duty it is to keep joints down to ¼ inch with cement mortar, will not have an enviable task.

Fifth.—Grout.—The Observatory specifications did not recognize this material. The author mentions it because the use of it was strenuously urged by workmen who had been employed on other Government works. Their practice had been in brick work, to lay up the outer courses of a wall in mortar, and then throw mortar in the trough thus formed, following with a libation of water from a watering pot until the mortar was converted into grout; the brick were then filled in. The practice may have some advantages in point of celerity in executing work, but assuredly not in giving the greatest strength and security to it. Grout was formerly much used in masses of stone masonry; thick walls with interiors composed of large and small rubble stones were treated by laying the outer faces in mortar, sealing up the joints and then pouring in grout until all interior spaces were well filled; this practice has become more honored by engineers, in the breach, than in the observance,

and is restricted to such parts of the interior of walls as cannot practicably be served with mortar—such as close vertical joints of heavy courses, etc.

The author made a few experiments, such as limited time and means would permit; they could not be pursued to conclusive results, but may not be out of place. At the commencement of operations sand of good quality for mortar was not available; it contained so large a proportion of earthy impurities, as loam, mica, red clay, etc., that it could not be used without washing. A better quality was sought and was obtained from a greater distance, being from the drift of the Potomac River. To avoid the expense of washing, an experiment was tried to ascertain to what extent impurities might be admissible. A sample of well-washed sand was taken and tested in mortar of two parts sand to one of cement; next, a sample with the same cement and an intermixture of sand with 18 per cent. of clay, and the result was, curiously enough, that the latter gave slightly more tensile strength than the former; this might have been accidental, though similar instances have been reported by experimenters. Without elaborating, it was concluded that good sand might be tolerated when not containing more than 10 per cent. of ordinary impurities.

For "second set" several trials were made—not that they were considered necessary, inasmuch as the matter is believed to be safely settled by existing knowledge in regard to it. Mortar that had taken its initial set and was remixed with the addition of more water, was found to lose about one-half the tensile strength due to it if used in proper condition. In the case of domestic cement there was but a trifling gain in strength in seven days over the strength of one day. With Portland cement the gain was better; it indicated a better result in time than the domestic. More experiments in this respect are desirable.

Mortar made at different Temperatures.—From the trials made it could not be found that there was any material difference in quality, whether mixed in a temperature of 50 or 60 degrees, or a temperature very near the freezing point. Some cold mixtures showed a trifle better than warmer ones. Some stress is laid upon the question by experimenters, and it seems desirable that the investigation should be extended so as to settle many points that arise under the necessities and ramifications of practice with mortars in our climate.

Deterioration by Age.—A test of domestic cement showed that the

time of set was doubled when one year old. Tests made of a barrel of Portland (date of manufacture not known) showed in an interval of time of nine months between tests, 80 per cent. increase in time of set, but no loss of strength between the seven-day tests of the two periods. Was the latter an accidental result?

Testing Cement.—The rules formulated by the able Committee of the Society upon this subject have been of great service, but there are a few points in connection with them which the author thinks many engineers would like to have more fully elucidated. It is mostly a question of manipulation and machinery. In order to make tests that all manipulators can compare with each other, it is important that they have the means of producing uniform and unvarying results. This, unfortunately, is not the case. Tests of one party differ from those of another, when made on exactly the same cement. The trouble known as the "personal equation" comes in and no practicable means have been devised to eliminate it. It enters into all the processes of mixing, applying water, filling moulds, immersing, the use of the testing wires, breaking briquettes, timing operations, etc.

The author has had some perplexing questions arise (as doubtless have others) from differences between his tests and those of experts who have operated with the same brand of cement. A decision affecting the quality and acceptance of a brand must sometimes be made under these circumstances. In one case that became questionable it was found that an expert had departed from the Society rules, in the one point of quantity of water used, and the tempering prescribed for his sample. This is believed to be the cause of many discrepancies that occur. From a large number of comparisons made by the author, in his practice, of the results of testing the same brand of cement under the same circumstances, the personal equation gives a considerable percentage of uncertainty as to the true quality of the material.

The mechanical devices in use are not as perfect as it is believed they can be made. A clutch which was used by the author for some time gave a fracture of the briquette at a point not at the weakest part, but at the sides of the briquette that were impinged upon by a too narrow point of the clutch; one-half or more of the briquettes developed this fault. He substituted another clutch with broader surfaces in contact and had very few failures afterward, in the proper fracture of briquettes at the narrowest point.

These difficulties have of course not escaped the observation of other members of the Society. They show the necessity of the exercise of much care and good judgment in passing upon the quality of cement. The author took occasion, for the purpose of a check, to follow up some tests; through the practical results obtained in construction the quest was somewhat involved; it embraced not only the contingencies due to the personal equation, but the shortcomings of mortar makers and masons already alluded to. The tests were desultory and inconclusive for the want of time and assistance in conducting them. With two aids as inspectors, whose time was taken up with more important duties, but little attention could be given to experimenting. Mortar was taken from the work as it was being used, the previous instrumental tests of which, together with the subsequent operations it had passed through before reaching the work, time, etc., being noted; it was then tested and the results compared with the previous ones. It must be said they were not reassuring or satisfactory. Without going into details it may be said that the set was very much prolonged and the tensile strength was impaired from what it ought to have been. What was previously remarked as to the habits of mortar mixers and masons the author thinks principally accounted for the results. It was a spur to greater vigilance as well as an illustration of one more of the contingencies that beset the pathway of the conscientious (shall we say overconscientious?) engineer in his efforts to insure good work. Tests of this kind will probably surprise those who make them in a thorough manner.

Some builders still use cement without instrumental tests as at present in vogue. By what means they determine the quality of the material the author is unable to say. Some years ago, as for instance, during the building of the old Erie Canal, and later in the enlargement of the canal (of which latter work he had some cognizance as a younger man than now), the tests were about as follows: For ascertaining the presence of quick lime in cement, a specimen was tasted, and also tried by a small ball mixed with water, which, after remaining a few minutes in the air was immersed in a tumbler of water; if it did not effervesce and immediately fall to pieces it was supposed to be free from lime. Also, if it kept its shape and showed signs of hardening in a few days, it was regarded as a favorable symptom of its strength. A further trial was made by placing mortar (usually mixed two to one) between two spawls and immersing in water; if at the end of two

or three days the mortar had not dissolved and run out, but showed a light film or crust on the surface, indicating the process of induration, it was held as safe for use. Fineness was mostly determined by rubbing a pinch of cement between the thumb and finger. Besides this, barrels were looked after to see if they were not damaged by exposure, causing discolorations, lumps, etc. It should be added that pats were made of cement which were broken after time was given for a full set. The author does not recollect that much cement was rejected. Later tests of General Totten and others followed, employing the loaded wires, and other means, breaking parallelopipeds, etc., until present methods were perfected for which due credit should be given to General Gillmore and other members of our Society.

Evidences of the quality of work are sometimes referred to by labor-saving persons, after the work is done, by probing mortar with the point of a knife, driving a nail into a joint, etc. Such proof may be satisfactory (supposing it to be successful) when applied occasionally as a check during the progress of work, but it is scarcely reliable and may be very misleading if not antedated by better tests.

Concretes.

Several experiments were made with concretes—difficulties being encountered with cement as before noted. The materials were, mainly, Portland cement, good sharp sand, broken stones (the latter specified to be "not over 2 inches in any way") and clean gravel. The broken stones, when not clean, were required to be washed; they were from sound gneiss rock, with some fragments of white quartz. Mixing was thoroughly done with careful attention to the use of water and tempering. As used in construction, batches were made with one barrel of cement, and the concrete was deposited in layers of 6 to 8 inches in thickness, and rammed until a show of water appeared at the surface. The experi-

The author has alluded to some old fashioned modes of testing cements which prevailed during the construction of the New York State canals. It will be proper to note the character of the works executed. Although the better modes of testing cement now in vogue were not then employed, yet the care and good judgment exercised by the engineers enabled them to construct some of the best specimens of hydraulic masonry that are to be found in this country; the locks and aqueducts of the eastern division of the Erie Canal are notable examples of this character. The eastern division—from Little Falls to Albany—is spoken of from personal knowledge, the author having commenced his engineering experience on the "enlargements" of that portion of the canal under John B. Jevois and William J. McAlpine.

Other portions of the State works exhibit equally good construction by other engineers of the period. Those old Romans of the profession have since joined the great majority, and have been gathered among the old builders who "builded better than they knew."

ments were limited principally to combinations of different proportions of ingredients and noting the results. They afford nothing particularly new, but may add something to the common stock of knowledge which we possess in regard to this important material. The author submits briefly and in condensed form, the prominent results.

The study of the best proportions and combinations of ingredients for strength and solidity, and at the same time keeping an eye to economy, is one of well-known importance. Uniform rules cannot be established, for the reason that materials are variable and cannot be obtained in all cases of equal quality; experiments can therefore give only approximate aids in the matter, showing what results may be obtained under stated conditions and circumstances. The author regrets that he is unable to give results of more value, such as might be obtained by the use of machine-mixing and machine-broken stone, etc. Trials were made with blocks formed in moulds of 12 x 12 x 6 inches, containing one-half of a cubic foot; some of the results shown were also obtained from large quantities of concrete as used in the work. It is absolutely necessary in making and using concrete, to have it under the eye of an inspector.

The following tabular statement sums up the chief points resulting from these experiments. The initial letters in column 2 are: P for Portland, D Domestic cement, S sand, Gr. gravel, St. broken stone; and the accompanying figures, as P 1, S 2½, etc., give the different proportions of materials used.

It was mentioned that a special description of concrete was required in the Observatory work. The purpose of this was to meet an astronomical necessity of obtaining a homogeneous mass of material in monolithic form that would be stable and subject as little as possible to atmospheric and other influences. Trouble had been caused in the old Observatory by reason of the telescope resting upon a foundation that proved a failure in these respects; the vision was disturbed by changes that occurred in the materials composing the foundation, caused, as supposed, by different temperatures and other conditions of the atmosphere. The base was a structure of stone masonry. It was proposed at first to construct the new work of béton, but it was settled to make it of concrete, as a cheaper material and equally good. Four separate foundations were required, in as many separate buildings. The drawing (Plate LVII) will illustrate the form and dimensions of the work. The buildings in which they were enclosed are shown as numbers 1, 2, 6 and 7.

1. No. of Sample.	2. Materials and Proportions.	3. Age of sample when tested. Months.	4. Weight of half cubic foot after a long set, as per Column 3. Pounds.	5. Time immersed for full absorption. Hours.	6. Absorbed per cent. of weight of Column 4.	7. Weight per cubic foot when saturated. Pounds.	8. Voids per cent. of volume after long set.	9. Quantity of concrete made from one barrel of cement. Cubic yards.	10. Cost in work per cubic yard.
1	P, 1; S, 2½; Gr., 3; St., 5.	5½	67¼	1½ to 2	.030	139	.030	....	....
2	do	4½	68½	2 to 2¾	.029	141	.064	....	....
3	do	4½	67¾	2 to 3	.020	138½	.048	....	....
3½	do	5	73⅓	....	....	....	....	....	....
4	P, 1; S, 3; Gr., 4; St., 7..	7	68¼	2 to 3	.037	141½	.080	1.65	$4 67
5	P, 1; S, 3; Gr., 3; St., 7...	8	71½	1 or less	.028	147	.064	1.36	5 50
6	P., 1; S, 3; Gr., 4; St., 7..	5½	72¼	1 or less	.028	148½	.064	....	....
7	do	4¾	69¼	1	.029	142½	.064	....	....
8	P, 1; S, 3; Gr., 3; St., 7..	8	70	1 or less	.025	143½	.056	1.36 to 1.44	....
9	P, 1; S, 3; Gr. 5.....	3½	69	3	.043	144	.096	....	....
10	P, 1; S, 2½; Gr., 3;St., 9..	9	66¼	1 to 1½	.030	136½	.064	....	....
11	P, 1; S, 3; Gr., 3; St., 8. ..	9	70¼	1 or less	.032	145	.072	....	....
12	D, 1; S, 2½; Gr., 3; St., 9.	8½	65	do	.054	137	.054	....	....
13	P, 1; S, 2½; Gr., 3; St., 4½.	....	....	....	....	....	....	....	6 87
		Some Formulas used on other Works:							
A	P, 1; S, 2; Gr., 3; St., 4..	Washington Monument.							
B	P, 1; S, 3; St., 5.....	do							
C	D, 1; S, 2; St., 4.....	do							
D	P, 1; S, 3; St., 5.	Prescribed by District Commissioners, D. C.							
E	do	do by Architect of Congressional Library.							

Nos. 10, 11, 12 and 13, above, not much used; No. 9 was for roof.

The foundations, as will be seen, were deep; the material in which they were excavated was a firm clay, with some gravel irregularly interspersed through it. It was supposed that a depth of 12 feet might be required for the concrete, but a good foundation was found for it at a depth of 9 feet. The pits were dug with perpendicular sides, and the concrete was put in, leaving a space of 6 inches around it, which was packed with clean sand as the work was carried up. The object of the sand was to relieve or check such vibrations or jars as might from any exterior cause reach the concrete mass through the surrounding clay.

In order to make a concrete as homogeneous as possible, the following mode was adopted: The cement was of a good quality of Portland, but it was impracticable to procure a brand that was even and uniform in set and tensile strength. Each barrel was tested, and those from the lowest admissible quality and upward were accepted. Then, as it took ten barrels to make a complete layer of concrete 6 inches thick, each ten barrels used were spread out on a broad platform and thoroughly intermixed. This produced a uniform quality for each separate layer. It was impracticable to carry the process of intermixing beyond this; it secured the homogeneousness purposed for each layer in itself, which was the best that could be done. After intermixing the cement it was measured back into barrels for use. For greater assurance as to the success in intermixing, samples of the mixture were tested and compared with the average tests previously made of each ten barrels. The tests of tensile strength corresponded in the two cases within 2 per cent. The mean of several tests gave a strength, neat, of 430 pounds. Broken stone of uniform quality was selected, but as it was not regular in size the largest were thrown out, and all portions of the size of ordinary gravel, together with all crushed and finer material, were screened out. Gravel was taken to supply the place. The proportions of materials were:

1 part Portland cement, 2½ parts sand, 3 parts gravel, 5 parts broken stone. Tests of this formula showed a filling of voids within 6 per cent. of the whole volume.

In carrying on the work each layer was completed before commencing another, and time was afforded for a layer to attain a strong set before working above it, and before depositing a succeeding layer the surface of the preceding one was sprinkled over with a film of dry cement. No layer was left unfinished over night, and the work was pro-

tected when left for the day, or when it was intermitted on account of weather. It is to be observed, that owing to the variable qualities of the cement, there might have been (without the precaution of intermixing) in various and irregular portions of each layer of concrete differences in strength amounting to 40 per cent. The weight of the concrete, as determined by many samples in blocks of 12 x 12 x 6 inches, was, after a long set, and when quite dry, 136 pounds per cubic foot. One barrel of cement of 380 pounds net, made 1.18 cubic yards of concrete. The method adopted of intermixing each ten barrels of cement prevented any material difference in strength between the layers. The film of cement between the layers was found by experiment to combine them thoroughly together into one inseparable mass.

The cost of intermixing was twelve cents per barrel, and the cost of all was $6 per cubic yard. This applies to the main blocks before the supplementary small piers were added. These piers, five in number on each block, are indicated on Plate LVII; they were truncated in form and 10 feet high, capped with granite, the tops reaching the floor level of the enclosing building. The buildings were of corrugated iron, this material being used as well for fire-proof purposes for security of very costly instruments, as for the purpose of enabling as uniform a temperature as practicable to prevail inside and outside the structures. No provision could be made for artificial heating, as the astronomical observers have to dispense with such conveniences without regard to the weather.

A good many trials were made with concretes besides those given in the previous tabular statement, but they are omitted, and such only are presented here as had reference to the work as executed. It will doubtless be seen, on comparison, that more economical formulas are adopted on some other works.

The object in the publication of these notes will be attained if a few practical results shall grow out of the effort. A glance at the results desired, and the necessity of obtaining them, will be proper in conclusion. The author has before him a list of failures and disasters which have occurred in public and private buildings within a few years past, in which many millions of dollars were involved, as well as some loss of human life, all traceable directly to defective construction, bad workmanship and bad materials, mortars and cements conspicuously included. The author does not include cases where the omission of

mortar and lax supervision have been disastrous. It is unnecessary to cite examples or to extend the catalogue. This can easily be done by all observing builders; non-professional individuals are only too familiar with them. As for instance, let them be asked to recount their costly experience in one form or another in remedying faults of construction, repairing defacements due to yielding foundations, such as cracked walls, derangements of plumbing and other interior work in buildings they have occupied and trustingly purchased, and if the list is footed up the sum total will appear in a convincing form. We cannot shut our eyes to the Protean forms in which these things occur. They are an offense to all good workmen, to all good builders, engineers and architects. Is it practicable to avoid them, and by what means? We deal here with the so-called "soul of masonry"—mortar. How shall we begin?

The author makes a few suggestions with deference, much preferring that they should come from other members of the Society. He would propose to start at the origin—with the manufacturers of cement. Cannot these be induced in some way to produce a material of less variable quality and of a higher standard? Imported cements could be labeled so as to give a reliable indication of their quality, date of manufacture, etc., with a guarantee of tests they will bear, say up to a stated age. If this cannot be done, then let the product be accompanied with a scale of prices for their agents, proportioned to its real value after the consumer has been at the pains and cost of laboriously testing it. Our domestic cements should be similarly treated; no one will doubt the practicability of doing it. We have the raw material in superabundence in all parts of the country; with proper care by experts in selecting, burning and grinding the material, an excellent quality can be obtained. Results have shown that when this is done we need have but little occasion for the use of the foreign material. Except for a few special purposes, the author believes we use much more of the latter than necessity or economy requires.

It is quite certain that if no complaint is made by engineers and architects there will not be much improvement. If builders in general will consent to the use of the promiscuous and variable brands that are found in our markets, without discriminating tests, and without seeking a higher standard, the standard will not be raised.

In testing cements there is a good opening offered to skillful inventors in the improvement of mechanical and other devices for proving with

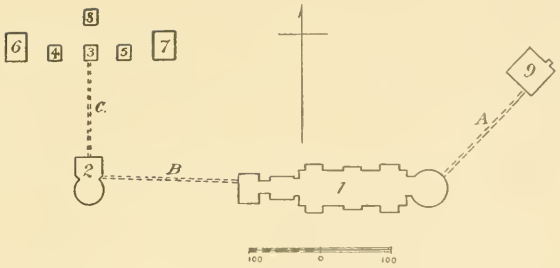
less doubt and uncertainty the value of cements and mortars. The "personal equation" should be eliminated. A valuable rule would be such an one as would govern a variable practice of accepting tests of one day, seven days, etc. Also, as to the acceptance of cement in large lots after testing one barrel out of ten, or twenty in one hundred or more. Such tests and acceptances it is believed are not uncommon. The author has been much puzzled in trying to apply them with a due regard to the doctrine of chances. In mortar making there is room for improvement. It is questionable whether all mortar and concrete should not be required to be machine-mixed when used in more than certain small quantities.

Grout, Wetting Brick, etc.—The mode of laying brick would admit of some useful rules in regard to practices which have been mentioned, especially in the use of grout. The ordinary method of wetting brick is deceptive; by throwing bucketfuls of water over a large pile, it reaches a few brick on one or more sides and escapes many. When it is known that a dry brick will absorb 6 to 10 per cent. of its weight in water, and requires four or five minutes for the process when immersed, the futility of common methods or subterfuges can be realized. Immersion is the only sure method to avert the evil consequences that are well known. These are only mere hints. More could be, but probably more than enough have been given. The author trusts he has not exaggerated their importance, or unduly pressed them upon attention.

To prosecute investigations such as have been alluded to requires not only labor and skill of a high degree, but an outlay of means for laboratory and other appliances, which latter necessaries are not a part of the endowment of the Society. But few engineers amass fortunes in the practice of their profession, although ample qualifications in other respects are not wanting, and it would be an onerous tax upon them to be required to explore exhaustively all the questions of scientific interest that are presented to them in their calling. Testing machines of great utility, involving large outlays, have been made by outside parties, and with Government aid, for the advancement of knowledge in the use of iron, steel and other building materials, and liberal contributions have been made by philanthropic individuals to the Society for other purposes. We regard these of course in an appreciative spirit, but may we not hope that the good work is not yet ended?

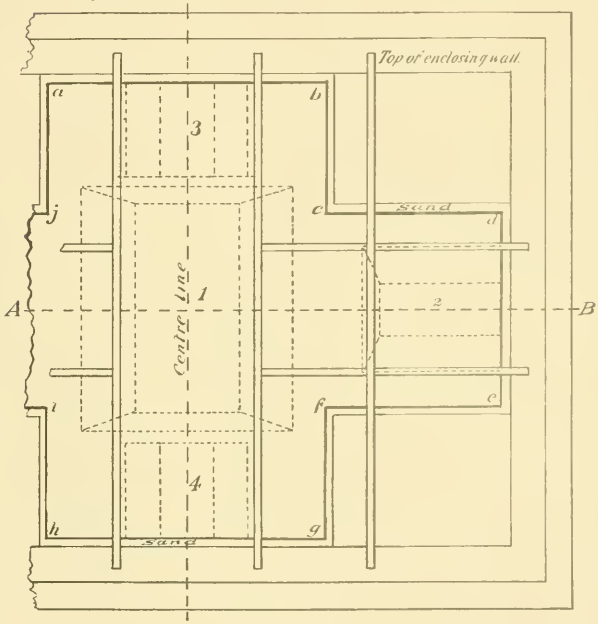
The author recently met with a remark which is as applicable to this

General Plan.

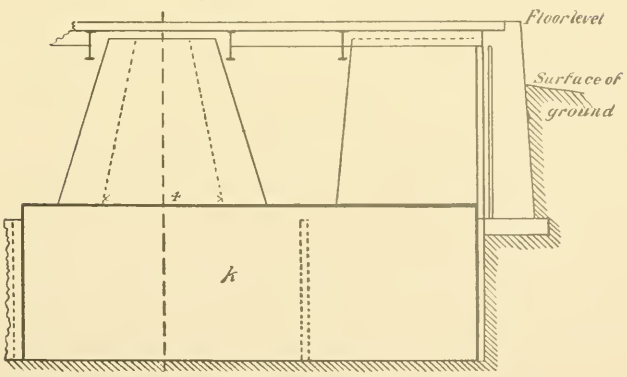


Part Plan of Concrete Foundations.

Heavy lines, outlines of Main Block as a.b.c.d.e.f.g.h.i.j.k
Dotted lines, on Plan, outlines of Supplementary Piers 1.2.3.4 &c
Light lines, other details.



Section on A.B.



topic as to the cause for which it was so fittingly made. The writer* was referring to the propriety of public and private aid in prosecuting bacterial investigations, and says:

"It is a pity that some of the large-hearted men in this country in whose hands great fortunes are placed for a little while, do not see to it that suitable endowments of research in this most promising field are furnished here." * * * "There has never been a time in the world's history when well-directed and well-sustained scientific research promised so much of positive and direct benefit to man as it does here in this under world of life to-day."

DISCUSSION.

J. Foster Crowell, M. Am. Soc. C. E.—I have nothing to offer of my own that is of any value, but I think the author of this paper is entitled to a great deal of praise for the way in which he has treated the subject. He has gone much deeper into it than usual, and has indicated one of the greatest difficulties that engineers encounter in securing perfect work. It makes no difference how good the cement may be or how well tested, if it be not combined properly with the other materials. The author has suggested that all mortars should be mixed by machinery, and that is one way of saying that the mixing of mortar should be looked after quite as carefully as the manufacture of cement. He speaks of the difficulty due to the personal equation in testing cements. I don't suppose anybody could ascertain the personal equation of the average mortar mixer, on whom the engineer must depend—in ordinary contract work. It seems that the Government or other parties having such very important ends in view—as the author describes in this case, should deviate from the contract system in this kind of work. A contractor cannot be reasonably expected to follow such rigorous specifications, and the engineer will always have difficulty in getting by contract the degree of excellence required by them. I think that one remedy for defective work in masonry would be found if the specifications covered all details of mixing as well as of material; when this is done, and tests are applied to the mortar used, I think we will begin to get much better work than we can secure while we confine ourselves to rigorous cement specifications alone.

James Owen, M. Am. Soc. C. E.—There is one very important question in the setting of cements, namely, that of time. Take a mass of masonry, say 7 or 8 feet thick, I would like to know how long it will

* "Glimpses of Bacteria," by T. Mitchell Prudden, M.D., in *Harpers' New Monthly* for April 1st, 1891.

take for the cement in the bottom of such a wall to arrive at its ultimate setting point? Some years ago I had to examine a masonry wall that was poorly laid, and I took some samples of the mortar from the interior. The wall had been built about four months, and the mortar was so poor that I condemned the wall. I took this sample of mortar to my office, and put it in my desk; some three or four years afterwards I came across it, and it was thoroughly hard and perfect. That shows that the question of time should enter in. In another case where I had to examine a mortar that had been in a wall some seventeen years, I found that it had been water-logged and had never set at all. It seems to me that the specified requirements for a cement should be adapted to the class of work that the cement is to be used for. The cement used for water works, for example, should have a more rigid standard than for ordinary masonry. The most important point is the question of time, how long will it take the cement to set, so as to give the strength required in any given case?

Charles B. Brush, M. Am. Soc. C. E.—It is perfectly true that there are remarkable features which cannot be explained in some grades of cement. It is also perfectly true that manufacturers start out and make a very excellent grade of cement for a long time, but do not always adhere to that same high grade. I believe it is true, however, that if an engineer has the reputation of insisting upon a certain grade of cement, and if he will deliberately keep on rejecting the cements submitted until he gets the grade that he wants, my experience is, it will save him a great deal of trouble, and the manufacturer also. I do not mean that he shall stand upon any petty point, but he can prescribe the grade of cement, and by insisting upon it, that grade will be furnished him. In the Dock Department for a long time all kinds of cement were furnished, until the Department decided what grade of cement they would take, and since then that grade has been furnished. If it is felt that from fear of delaying the work, or from complaints of that kind, the engineer will give way, I think he will always have a great deal of trouble in getting a proper cement. A great deal depends upon the firmness of the engineer in charge of a work.

The Chair (A. Fteley, Vice-President).—What is your opinion in regard to the tests of neat cement as against tests of cement and sand?

Mr. Brush.—I think both tests should be used. I do not think you should rely upon the one or the other; I do not think you should take any one test in regard to cements. As to the question Mr. Owen has raised, how long a cement keeps on setting. Some cements set more slowly and some more rapidly; that depends on the mode of manufacture and ingredients. I do not think either neat tests or sand tests alone should be relied upon; I think both should be taken.

The Chair.—In work which requires a very large amount of cement, and where, in justice to the contractors, it is necessary that the cement

should be tested properly and an answer given promptly to the contractor—do you not think that the quick test of twenty-four hours with neat cement is highly desirable ; not because it is the best test, but because, having established a standard for your cement by such a test, you can tell that the cement which you want will, after twenty-four hours, give you so much, and by repeating that test on a large number of barrels, you can tell whether you will take it or not ?

Mr. Brush.—I do not think there is any question about it. I think that as soon as the cargo comes, the contractor should be informed what the result is as to that cargo. If the same person has made all the tests, as should be the case, it is really a question of comparative results. If the results during that twenty-four hours are favorable, the contractor should be informed so that he may proceed, subject to later tests ; a contractor should not be held back with a large cargo.

I would like to hear from Mr. Myers, who has been giving a great deal of attention to the subject. I know that a great many different grades of cement have been offered to the Park Department and that they had some difficulty in relation to them ; but my impression is that by firmly adhering to the position that he would have a cement which was satisfactory, he always got it.

C. H. Myers, M. Am. Soc. C. E.—I can only say that we always required the cement to reach our standard, and if it did not show according to the ordinary tests that it was up to that standard we rejected it, and other cement had to be furnished which came up to the standard. Sometimes a brand that had been giving favorable results would not answer the tests ; we concluded that there was some defect in that particular lot and rejected the lot ; at another time the same brand might answer the test at once. Sometimes a new brand would be furnished and the same procedure followed until it gave the cement we wanted.

Mr. Brush.—Did you not always succeed in getting the grade you wanted ?

Mr. Myers.—We always did, and I might add that, as an indication of the ability of cement makers and dealers to supply a required grade of cement when effort is made to do so, we experienced less difficulty in obtaining what we wanted after the requirement for tensile strength was considerably increased than we had had previously.

F. Collingwood, M. Am. Soc. C. E.—I think from what Mr. Brush says he would qualify it by saying that all cements should be subject to a long time test sufficiently often to be sure that it is sound. Some cements will give great strength in a short time, but lose strength afterward.

Mr. Brush.—I understand that. The long time tests would prove the results. I would like to have our Chairman give us some of his experiences.

The CHAIR.—I beg to be excused from giving my views on this subject for the present. One of the troubles with all cements is the presence of free lime, and in this connection I might mention a fact which I have seen in a published report by Mr. George F. Deacon in connection with the building of the Vyrnwy dam. Portland cement was used there, and Mr. Deacon, having recognized the presence of free lime in almost all the Portland cement that he bought fresh from the factory, found it necessary to organize a system by which he could get rid of it. In order to do that he built a large frame building, with seven galleries or shelves all around on the inside. The floors of these galleries were formed of movable slats, such as we use for blinds; these were each moved by a single lever at the outside of the building. Supposing these floors to be covered with cement, each floor holding about one day's supply; the first used would be always taken from the lower shelf; at the end of the day, by opening the slats, the cement on the second shelf was dropped on the lower, the cement on the third shelf was dropped on the second, and so on, leaving the top shelf empty and ready to receive a load of fresh cement. In that way the cement had been handled and exposed to the air seven times before it was used. Every day the amount of heat developed by the mixture of cement was recorded, and it was found that each day of this treatment lessened the temperature of the mixture about one degree until it was finally acceptable for practical purposes.

Mr. COLLINGWOOD.—This process of seasoning cement is very fully described in the specifications for a bridge across the Seine at Rouen, in which it was provided that the cement was to be delivered in certain quantities at certain times, and spread over a large floor, protected from the weather, there to remain long enough to thoroughly slake all free lime. None was allowed to be used without undergoing this exposure to the air.

ARTHUR S. C. WURTELE, M. Am. Soc. C. E.—The paper of Mr. Grant is a description of the work of construction of the Naval Observatory at Georgetown, showing the troubles of the engineer in having work done up to specifications. His remarks on cement testing show clearly that the specifications in that matter were unreasonable, in that a very slow setting cement was required and at the same time excessive one-day and seven-day tests; so much so that the requirements of the specifications were finally abandoned to enable the work to be completed. A curious and unjustifiable difference was made in prescribing a lower test for Potomac cement than for Rosendale. Several experiments were made, but the figures were not given, except in a general statement of results which must be indefinite from the small number of experiments. I take exception to the statement that 10 per cent. of clay may be allowed in sand without detriment, and that result seems to be given on a single experiment. In State work I do not consider less than four-day tests,

and I find when they give over fifty pounds breaking strength, with the briquettes clean and smooth, the cement invariably comes up to the specifications for seven and twenty-eight day tests. Less than four-day tests are liable to be very variable.

Mr. Grant objects strongly to a variation in the weight of barrels of cement, but I do not know that they are to be expected to be uniform in weight, and attach no importance to such a difference. He also objects to grout. Now my experience is that masonry properly grouted gives better results than without the use of grout. As to the personal equation, the State practice is to have all testing done by the same man and thereby eliminate error on that account, as the error if any is the same in all cements. It is not surprising that mixing quick lime with cement was a failure, but I do not understand how the temperature of the water used made no difference, and think the result is due to too few experiments.

The table of concretes is valuable as showing the results of different combinations. The mixing of ten barrels of cement together for concrete would undoubtedly give a more uniform concrete and is a good precaution where exact results are required. The only way to improve cement is to reject all that does not come up to a fair specification. If a working specification could be settled on it would be advantageous to manufacturers, but it would have to be entirely reasonable, as shown by the abandonment of the specified tests in this case of the Observatory.

The remarks about wetting brick before use are quite correct. Complete immersion of brick is absolutely necessary for good work; sprinkling or throwing water over them is of no use.

D. J. Whittemore, Past-President Am. Soc. C. E.—We can assure Mr. Grant that his experience in using and testing cement mortars is but a parallel to that of many of our members who have conducted similar work and tests. I take it for granted that manufacturers of cements, both foreign and domestic, endeavor to furnish a product both good and uniform in quality, and I can readily surmise why they desire the tests to be conducted at their works, as experience is that results there attained are exceedingly liable to vary to an extraordinary degree from the tests conducted at the place where the cement may be used, even when the tests are similarly conducted at the two places.

Cements often deteriorate in proportion to the number of middlemen through whose hands they pass. The increase in number of such between the producer and consumer indicates added age to the product and sometimes something worse. It is not unknown that some middlemen have occasionally mixed good and bad cements, or even Portland with natural cements, and placed the mixed product on the market under the brand of the best. It is a pretty well established fact that foreign Portland cements deteriorate markedly in transit by ocean to this

continent, and this continues to occur during transit by canal, lakes and rivers to such interior ports as are thus reached and supplied.

I have it on good authority that occasionally, when cements reach their destination in a bad condition, so much so as to be hard, lumpy and difficult to shovel from the barrel, the middleman, after making such settlement as he can for a rebate from the consignor, regranulates or repulverizes the defective cement and places it on sale under its original brand. An instance of this kind brought to my knowledge is as follows : A vessel bound for Chicago loaded with foreign Portland was partially wrecked and the barrelled cement was well wetted. These barrels were delivered on dock on arrival, and were not accepted by the party for whom they were intended, and finally were given to a contractor as compensation for their removal. As much of this cement as could be conveniently pulverized was so treated, sold and used in the market mentioned, as genuine Portland. Another instance that has come to my knowledge within the last few days: Of a cargo of foreign Portland received this season at New Orleans, one hundred barrels were shipped to an important city up the Mississippi and there received by the importer. Sixty of these barrels were found caked, but fortunately the importer was honest and would not allow it to be repulverized and placed on the market for sale.

It is certainly gratifying to our members to notice the complimentary remark at the commencement of Mr. Grant's paper relative to the rules laid down by the Committee of our Society for testing cements; yet as one of the members of that Committee, permit me to say that I wish these rules were more satisfactory in some respects, and in the light of further experience it is my belief that they can be made so. What we fail to secure by these rules is the determination of the absolute degree of plasticity of the mortar to be used in the fabrication of test specimens, and also the absolute pressure required, or desired, in moulding.

Since the publication of these rules several ingenious machines have been devised for moulding, which are intended to secure (and do apparently) uniformity of manipulation. The indications are that Mr. Grant followed the rules laid down by the Committee of our Society closely, yet all experimenters do not do this. Several instances of this nature have lately come to my notice, of *tyros*, having charge of their first work in mortars, who appeared to have the notion that mortar was a nasty semi-liquid mass, and that the only work needed was to lift the same on the spatula or trowel and slush it into the mould, strike it off level and allow it to set with all the water and air spaces in the body of the briquette that such manipulation will permit, and then expect satisfactory results from the tests. Again, cement manufacturers soon determine how to manipulate test samples so as to secure the best possible results, making error in their direction, probably not in degree, that the *tyro* makes in his.

Some years ago my attention was called to what purported to be a very learned paper upon cement tests, published in an engineering journal (not ours) in which some astounding, unique and negative results were shown, and the graphic representation of the strength of cement mortars at different ages shown thereon could well be mistaken for a profile of the Rocky Mountains. It appeared that the cement mortars tested were first moulded and exposed, in the form of a long slab, having a cross-section somewhat the form of a briquette, and when tests were required briquettes of the thickness desired were cut off the slab with a hand-saw.

Opinion hastily formed, and based upon slovenly conducted experiments, is the bane of all scientific work; and it is safe to assert that the results of the first three months' work of a novice in cement testing should never appear in print.

In the report of the Committee on A Uniform Method of Cement Tests, experience has shown that an error was committed in attaching thereto the addenda giving the Committee's opinion as to what strength different cements should indicate. It may be proper to say that this addenda had the opposition of at least two of that Committee, and it was only through the united opposition of these two members that the matter was expunged from the body of the report, with the understanding that it might appear as an addenda, the publication of which has certainly been unsatisfactory to the producers of our natural cements, and misleading as to the strength of Portland when tested as advised by the report. This can be cited as another instance of the ill effects of reporting on matters beyond the specific duty delegated to a committee appointed by the Society. The fact is that at the time this report was formulated, little was known relative to the strength of Portland cements when test samples having a breaking area of 1 square inch are used. The bulk of the tests previously made were from specimens having a breaking area of $2\frac{1}{4}$ inches square.

In a paper read by me before this Society at the Annual Convention, at St. Louis, several years ago (*Transactions*, Vol IX, 329), it was shown that in tests of different sizes of specimens of the same mortar, the strength varied per square inch approximately in an inverse ratio to the periphery of the breaking section, that is to say that a briquette with a breaking section of $1\frac{1}{2}$ inches square ($2\frac{1}{4}$ square inches area), indicating a strength of 40 pounds per square inch, when tested in briquettes having a breaking area of 1 inch square would show a strength approximately of 60 pounds.

I trust our Society will reappoint a committee upon improved methods of cement tests, in the hope that a better solution of matters that appear indefinite in the former report will be reached, and at the same time methods of testing be devised which will not involve long time delay in determining results. It is quite clear to me that this can in a

measure be accomplished by the use of steam bath exposure to hasten induration, whereby activity and ultimate energy can with a reasonable degree of certainty be determined. This method of hastening activity was used by me some sixteen years ago, and the same process was then and is now successfully used by some cement sewer pipe manufacturers in expediting their work. Henry Faiji, M. I. C. E., one of the most accomplished cement experts of England, has in his London laboratory a device of his design for this purpose.

The varying results of Mr. Grant's tests indicate pretty conclusively that the Portland cement used was, in one respect, that which was called for by the architect's specifications, genuine imported Portland ; as some of it appears to have been subject to a tempestuous ocean voyage or to other equally beneficial treatment before it arrived at the place of use.

In conclusion, I wish to call attention to the fact that our Government exacts a duty on foreign cement, supposed by many innocent-minded persons to have for its object the encouragement of the manufacture of Portland cements in this country; but in this instance the architect, who is also by virtue of his office an agent of the Government, requires the foreign product, thus discouraging any attempt on the part of our citizens to produce such cements in this country ; hence I am forced to the conclusion that this duty is interpreted by the architect to mean *for revenue only.* Our people are paying millions yearly for the foreign product. It is now known to many engineers of the West that we are producing in the interior of our continent Portland cements equal in quality to the very best, and far superior to a large bulk of the foreign product brought to and sold in our country.

W. W. Maclay, M. Am. Soc. C. E.—Mr. Grant's remarks about the superiority of machine-mixed mortar over hand-made mortar are very appropriate, and his objections to the re-tempering of mortar and the use of grout will be cordially indorsed by most engineers.

In reference to the deterioration of Portland cement by age, I am sure, from experiments carried on for a number of years, that Portland cement deteriorates very slowly, a year or two causing practically no loss of tensile strength, although, of course, invariably making the cement more slow-setting. I unite with Mr. Grant in wishing to have some method of cement-testing adopted that will eliminate "the personal equation" of the operator. But while it is difficult to attain this result by our present system, by care and attention it is possible to largely reduce the probability of error, so that, for example, with a trained cement-tester the average differences in re-testing the same cement should not exceed 5 per cent. As I have said in another publication: "It must not be understood that the testing of cement is other than laboratory work, for all attempts to popularize cement-testing so that it can be brought within use of all who use it, have signally failed. Cement tests, to be of any comparative value, will always have to be made by

persons accustomed to accurate methods and possessing a degree of experience and skillfulness in this particular line of physical research, which is only to be acquired by long practice. No great equipment of scientific education is necessary for the cement-tester; but, on the other hand, he must have a large amount of accuracy and practical experience, with the conscientiousness that will prevent him from slighting or slurring over details in order to obtain results." There is one point in the paper in relation to the great want of uniformity in quality between the barrels in the same lot, as experienced by Mr. Grant, from which I beg leave to differ, for the following reasons:

In the manufacture of cement by the grinding and conveying processes, the cement is thoroughly mixed in large quantities, and the quality therefore varies in large quantities, in some relation generally to the quantity produced per day or per week. From the very nature of the manufacture it would be impossible to make cement, varying much in the different barrels; and from my experience I feel perfectly sure in the results of testing samples, taken from every tenth barrel in any cargo or delivery.

W. R. Hutton, M. Am. Soc. C. E.—The difficulty with the cements used at the Naval Observatory was probably due, in a great part, to the "small margin of profits" to the contractors, as mentioned by Mr. Grant. There should be no difficulty in obtaining either Portland or Rosendale cements which would exceed the very moderate requirements of the specifications. Although the Portlands are deteriorated by great age, yet a certain period of time must elapse between their manufacture and use, and there must be a certain exposure to air to slack such free lime as they may contain. The domestic cements, on the contrary, should be used while fresh. The very low results, given in the paper, of the tests of Rosendale cements are remarkable. As a rule, it is very uniform in quality, and gives good results in the testing machine.

The Potomac cements have been in use since 1830 upon the Chesapeake and Ohio Canal, the Baltimore and Ohio Railroad, and local works. Formerly they were of uncertain and variable character, depending largely upon the care used in the selection of the stone and its manufacture. During the last twelve or fifteen years they have come into vogue in Washington, and have there a good reputation. Some of the layers give a remarkably quick-setting cement, equal in that respect to the Vassy, and like it, requiring to be mixed by the trowelful, and used almost within the minute. But the uses of such a cement are few. Other layers are of a less active set, and give very good results when not overloaded with sand. For important work along the Potomac it has been used with one to one and a half parts of sand. In its original estimates for the Chesapeake and Ohio Canal, in 1826, the Board of Internal Improvement estimated for the use of "water lime" to be brought from New York at a cost of $1.38 per bushel, delivered on the work. It was

to be used only below water-line in the tunnel, and in the bottom and facework of the locks, the remainder of the work to be built with common lime mortar.

The admixture of common lime to prolong the time of setting cannot be adopted without some loss of strength, although for most purposes good work may be obtained with the combination, and with some economy in cost. On the other hand, the addition of a small proportion of cement to lime mortar increases greatly its strength and hardening properties. In the facework of Dam No. 5, of the Chesapeake and Ohio Canal, Rosendale cement was mixed with that from Round Top (Potomac), to delay the setting, and with good results.

No doubt, the "personal equation" enters largely into the results of tests of cement, chiefly as regards the thoroughness of the manipulation and the degree of compression applied in packing it in the mould. This latter condition is particularly well known to the agents of the cement companies, and perhaps accounts for the offer by one of them to guarantee results if tests were made in New York. Uniformity of results is obtained by the use of what may be called the German method, referred to in the paper by Mr. S. Bent Russell, and this may be useful, in the comparison of different cements; but the treatment is so radically different from that which the cements get in actual practice that it is of no use in determining the practical value of the material. For that matter the usual tests for tensile strength give only a collateral indication of the value of the cement. What we need is adhesiveness and resistance to compression. So the one-day test is useful to indicate the age and condition of the cement, uniformity in a given delivery, or in the output from a given mill; but low one-day tests are not incompatible with very good results after one or several weeks.

Arson (Construction des gazométres) notes that those Portland cements which contain the least free lime are the most impermeable to water. The lime in time dissolves, leaving the cement porous. They also set more slowly and give the stronger mortar. His practice with the Portlands is to mix them, three parts sand to one of cement, with an excess of water, and let them stand on the mortar board from one to four hours, depending upon the age of the cement. The cement rejects the surplus water, which drains off, and the mortar gradually stiffens; when it has become pasty it should be used. The walls of gasholders are built with this mortar, but to render them water-tight they are plastered on the inside with Vassy cement, mixed with equal parts of sand. The cement is tested for tensile strength and for impermeability. It is found that mortars which are kept in the air are more permeable than those set in water. That a troweled surface is necessary—water under pressure, of two to three atmospheres, being applied to the back of a disk penetrated readily as far as the troweled skin, and was there arrested. If well troweled and the connections well made, the mortar should be

impermeable under 40 feet head of water. Care must be taken, however, to prevent a pressure from the outside of the wall, before the mortar has set.

The method of making grout described by Mr. Grant is certainly unique. Although the use of grout is now justly proscribed, it was formerly a usual method of building heavy masonry. The specifications of the Erie Canal provided that the joints should be filled with grout. On the Chesapeake and Ohio Canal its use was general, as upon the Baltimore and Ohio Railway. Dressed stones were bedded in mortar; rubble was laid dry, and all was grouted full. There are cases where its use cannot be avoided, as sometimes in the repair of old masonry; it should be forced in under pressure, as was done in the repair of the new (New York) Aqueduct, and around Mr. Greathead's City and South London Railway—in the one case by pumping, in the other by the pressure of air. A better method of filling joints than with grout is to work the mortar in by means of a "sword," a thin blade of steel, preferably serrated. All the ringstones of the Pont Ste. Marie in Paris were set in place dry with spacing strips in the joints, and when all were in place the joints were filled with mortar, not grout, by means of a sword or fiche. The arch stones of Cabin John Bridge of the Washington Aqueduct were laid on spacing wedges and filled with grout.

To recall the old experiments on this point, Rivot and Chatoney made a concrete block by pouring a grout of Portland cement (1 cement 0.25 water) under a head of 17 feet, into a closed box filled with broken stone, and with satisfactory results both as to strength and the complete filling of the voids. Vicat thereupon filled tubes with mortar of ordinary consistency and others with grout, using different proportions of water, 50 and 100 per cent. He found upon testing that the grout was much weaker than the mortar, and even that the material in the top of the tube was less hard than that in the bottom. As a comparison this is not conclusive. Rivot and Chatoney used but 25 per cent. of water, and applied the grout under considerable head. Although experiments show those mortars to be the stronger which are made with the least proportion of water, yet in actual work it is better to have the mortar rather soft than stiff. Better and more solid work is made with it, and its adhesion is greater.

Boehme's experiments indicate that the resistance of mortars of Portland cements to compression increases for from seven to twenty-eight days, more rapidly than its tensile strength; at twenty-eight days it is about nine times as strong. (The Prussian rule is ten times.) The tensile strength of his specimens (3 to 1) at twenty-eight days was 293 pounds per square inch, which is a little less than half that of the pure cements at the same age.

Mr. Grant's experiments of mixing clay with the mortar is hardly conclusive as to the admission of the impurities usually found in sand.

These consist most frequently, not of clay, but of loam, mica and organic matters. The strength and impermeability of concrete depend largely upon its being properly and sufficiently rammed. The breaking of the briquette in the testing machine at the point of contact of the clip is remedied by enlarging the surface of contact of the clip or by placing a bit of leather between the surfaces.

A. P. Boller, M. Am. Soc. C. E.—The following tests upon 12-inch cubes of concrete were made to determine the strength of the concrete used in the monolithic foundations built for the 155th Street viaduct, New York City. This concrete had been made with great care, and proportioned of one part Portland cement to two of sand, and four of 1-inch broken stone. The cement used was the well known Alsen brand, showing an average tensile briquette strength of 450 pounds in seven days. The batches were mixed quite dry, the successive layers of concrete just flushing moisture to the surface, under ramming with 35 pound metal rammers.

The tests were made October 5th, 1891, on the 1 200-ton horizontal machine of the Phœnix Bridge Works, the compression faces between which the blocks were broken having a ball-bearing attachment to the ram and head block. All the blocks with the exception of No. 6 were broken at right angles to the planes of ramming, and crushing surfaces were smoothed and made true with plaster of Paris. In all cases failure was accompanied with two resulting pyramids, the larger one of which was always the lower part of the specimens as rammed in the box forming the cubes. Specimen No. 6 had a piece about 3 inches thick split off in the plane in which it was rammed, the remaining thickness of 9 inches being crushed in the direction of the plane, instead of at right angles as above. This specimen had been fractured in shipment and was accidentally broken, although, like the other specimens, it had been shipped to Phœnexville in a wooden box made from 2-inch plank. It might possibly have been broken in nailing up the plank around it.

Experiment No. 1, made October 4th, 1890. The block was one year one day old, mixed from cement one part, sand two, and of 1-inch stone four parts. It was taken from the platform as mixed for the work, and having just enough water to show moisture on the surface when well rammed; weight of the block at the time of test 153½ pounds; crushed at 2 030 pounds per square inch, or 146 tons. The bottom pyramid was 8 inches high, with 10 by 11 inch base. The upper pyramid was destroyed in removing from the machine, to which it adhered through the plaster. The fractures were clean and sharp, many through the broken stone (Tompkins' Cove limestone), and the surfaces of the cone not friable.

Experiment No. 2. A block made from the same batch as No. 1, and more water added, so that it could not well be rammed, and would flow if the box were tipped over; weight at the time of the test, 152 pounds; crushed at 1 591 pounds per square inch, or 114½ tons. The bottom

pyramid was about the same size as that of No. 1. The edges of the pyramid were well defined and the surfaces sandy and friable.

Experiment No. 3, made October 22d, 1890. Block eleven months fourteen days old. Taken from the platform as mixed for the work as for No. 1, with just enough water to flush the surface when rammed; weight of the block at the time of the test 155 pounds; crushed at 1 636 pounds per square inch, or 118 tons. In this specimen, while the bottom surface was smooth the top surface was irregular, the box not having been properly filled, which may account for the great difference in strength from No. 1, where the crushing surfaces were in close contact with the machine, enabling the pressure to be at once uniformly distributed, while the irregular top surface of No. 2 brought the pressure at irregular points, save aid in the distribution by the plaster of Paris. The larger pyramid was 6 inches high, with a base having two sides 9½ and 10½ inches, and two sides 5½ and 6½ inches. The smaller and upper pyramid was approximately conical, with a base 5 inches diameter, and height of 4 inches. The fractures were clean and sharp.

Experiment No. 4, same as No. 3, with water added, so that the mixture could be rammed with difficulty, and would flow only slightly if the box were tipped over. The age at the time of the test was eleven months fourteen days, and the weight 154 pounds. It crushed at 1 680 pounds per square inch, or 121 tons. The bottom pyramid was 5½ inches high with a base 10 inches square. The fracture was not well defined, and the surface was friable and sandy. The upper pyramid crumbled as before in taking it from the machine.

Experiment No. 5, made October 22d, 1890. Block eleven months fourteen days old, with a weight of 117½ pounds. The mixture was: cement one, sand five, and no stone, and it was mixed very wet, so that water came freely to the surface when it was slightly rammed. This crushed at 1 015 pounds per square inch, or 72¾ tons. About 1½ inches at one edge of the block had been broken off, probably in its removal from the mould box. The surface was quite rough and required a thick coat of plaster to bring it true. The resultant bottom pyramid was 7 inches high and as near as may be 11 inches square. The upper pyramid was about 2 inches high with an elliptical base of 6 by 9 inches and a fracture sandy and friable.

Experiment No. 6, same as No. 5, with just enough water to show moisture when well rammed; weight, 127 pounds. This block was split slightly concave, as before noted, into two pieces 3 and 9 inches thick respectively. The experiment was made on the 9-inch piece in the direction of the layers, or perpendicular to the plane of ramming, and the block crushed at 1 072 pounds per square inch, or at the rate of 77 tons per cubic foot. The fracture was sandy and friable. The specimen split up as if it were under pressure and left no pyramids whatever.

Experiment No. 7, made November 6th, 1890. Block eleven months old; weight 147½ pounds, cement one, sand two, 1-inch stone five parts, mixed with just enough water to show moisture when well rammed. This crushed at 1 370 pounds per square inch, or 98½ tons per cubic foot. It did not break in regular pyramids; the lower piece was of irregular shape 7½ inches high, and the base about 11 inches square. The fracture was clean and sharp, and the surface not friable. The small or upper piece was not preserved or noted.

Experiment No. 8, same as No. 7, with more water added, so that it came freely to the surface when slightly rammed; age of the block 11 months; weight, 154½ pounds. It crushed at 1 883 pounds per square inch, or 136 tons per cubic foot. Two pyramids remained, the upper 5 inches high and with a semi-circular base 11 x 9 inches, and the lower 7 inches high, with a rectangular base 11 x 11 inches. The fracture was fairly sharp and the surfaces friable.

In the above experiments, the mould boxes were well soaked before concrete was put into them, and the crushing record noted from gauge readings, no allowance being made for the friction of the machine—an undetermined quantity. Briquette records of the cement, taken from the barrels from which the blocks were made, showed the following:

No. of Experiment.	Neat.			Mortar, 1 Sand, 1 Cement.		Mortar, 3 Sand, 1 Cement.		Mortar, 5 Sand, 1 Cement.	
	7 Days.	3 Months.	1 Year.	4 Months.	1 Year.	4 Months.		4 Months.	1 Year.
Experiments 1 and 2	399	526	728	415	715	186	.....	.....	.....
Experiments 3 and 4	540	654	692	.....	.....	.....	.....	.....	.....
Experiments 5 and 6	.....	.....	.....	.....	.....	.....	.....	152	189

Cement 98½ per cent., fine, 2 500 mesh sieve, briquette mixed with 20 per cent. water and left one day in air.

William H. Grant, M. Am. Soc. C. E.—The general tenor of this discussion strengthens the view which was the origin of my paper—that of showing the necessity of establishing more definite rules in the treatment of cements and mortars. It is quite satisfactory in that regard, and I make my acknowledgments for the commendatory remarks. The dissenting portions of the discussion require further attention.

Mr. J. Foster Crowell speaks of the tests at the Observatory as rigorous, and says a contractor cannot be reasonably expected to comply with them.

Mr. Arthur S. C. Wurtele thinks the tests were unreasonable. He also speaks, in the same connection, of "a curious and unjustifiable difference made in prescribing a lower test for Potomac cement than for

Rosendale." A little care in reading my paper would have shown that this latter remark did not apply to the Observatory, but to tests for municipal work in the City of Washington. As to the tests for the Observatory work which I have described, I took some pains to compare them with the requirements on other public works, with the tests which manufacturers guarantee for their brands, and the tests which many trials by experts show that cements, both foreign and domestic, easily bear; and I cannot, therefore, think that they were unreasonable. That a good deal of cement which was brought to the work proved of inferior quality is not to be taken, as most engineers, I think, will admit, as showing that the prescribed tests were too high.

Mr. Crowell thinks that if specifications covered the details of mixing (and tests) we would get better work, etc. I agree with him that it would be desirable to specify clearly the requirements and tests for cement in all cases, in order to avoid differences about them with contractors and dealers. This reply to Mr. Crowell may be briefly taken to cover Mr. Wurtele's objection to the cement tests. My tests were by no means so few as he thinks. I thought I had amplified the subject quite fully in my paper.

It seems to me he lays undue stress upon the circumstances which I mentioned of modifying the tests of domestic cements when only a small amount of work on top of the Observatory walls remained unfinished. I explained why, at this point, the Navy Department authorized the relaxing of the tests. If it was an extremely low requirement, but little harm could be done under the circumstances, and it was by no means an admission that the previous tests had been too high. In fact, a fair comparison of the tests for domestic cement (which was by far the largest in quantity used) with those required on other works, will show that they were below rather than above a usual and justifiable standard. As most of the members who participated in the discussion have passed over these points, I infer that they did not regard them as excessive.

Mr. Wurtele objects, quite properly, to any adulteration of sand. I mentioned an experiment I made with sand and clay intermixed, which showed an unexpected result, and I was inclined to attribute it to accident, suggesting that more experiments were desirable. I inferred from it, perhaps erroneously, that good sand might be tolerated when not containing more than 10 per cent. of ordinary impurities. In my experience purer sand than this is rarely found. Is it really important to incur the expense of washing such sand ?

My remarks as to the use of grout were objected to by Mr. Wurtele. He says that his experience is that masonry, properly grouted, gives better results than that without the use of grout. We all know that grout is simply thinly diluted mortar, and it is a well established fact that mortar loses its vitality in proportion to the excess of water used in it ; a further difficulty arises in its use, that of the sand and cement separating

from the water and gravitating to the bottom, or stopping short of the point it is intended to reach when used in any considerable quantity. I had some experience in the use of it on the New York State works, where it was quite a general practice, but have never been convinced that any work was better for it than if good mortar were substituted in its place. I must, therefore, leave the remark I made about it without further qualification.

It does not seem necessary on my part to extend this discussion, and I conclude by only adding that the interest manifested by members on the subject of my paper is regarded as a favorable omen that the suggestions made, and which have been especially so clearly enforced by Mr. Whittemore and Mr. Maclay, may receive the attention of the Society at an early day.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—September, 1891.)

NEAT TESTS vs. SAND TESTS FOR PORTLAND CEMENT.

By S. BENT RUSSELL, M. Am. Soc. C. E.

WITH DISCUSSION.

The matter which I have to present will be found in the attached table and diagram. A few lines will explain their meaning and show how the data were obtained. The cement tests, from which the table was constructed, were all made recently in the laboratory of the St. Louis Water Works Extension. They were made not for the purpose of comparing the methods of testing, but for the purpose of comparing the tensile strength and value of twelve different brands of cement. In order to make a fair comparison it was decided to test each cement in three different ways.

First, forty briquettes of each brand were mixed on the jig. In this test enough cement for one briquette is weighed out into a brass cup, the proper amount of water is poured in, a tight-fitting cover put on, and two of these cups are violently shaken by a special machine called the jig.* When thoroughly mixed the mortar is removed from the cup and pressed into the mould by hand.

* Plates LVIII, LIX, LX and LXI show general and part detail plans of a jig. It is driven by a small power at the rate of about six hundred revolutions per minute. One minute is usually allowed for mixing a pair of briquettes.

Forty briquettes of each brand were then mixed by hand in the usual manner with a palette knife on a mixing table. In both of the above methods of test the cement was used without sand, and such proportion of water was used as would give a standard consistency of mortar. This consistency was such that when ready for the mould, the mortar could readily be formed into a ball about 1 inch in diameter with the fingers, and rolled in the palms without breaking. No more water was allowed than was necessary to produce this consistency, and this result would only be reached after about three minutes hard and skillful working with the palette knife for each briquette. In both hand-made and sand tests each briquette was made independently. The proportion of water for both hand-mixed and jigged briquettes was different for different brands. From 20 to 23 per cent. of the weight of the cement was taken.

Lastly, forty briquettes of each brand were mixed with sand. The sand was thoroughly dried and sifted between the No. 20 and No. 30 sieves, and was of superior quality; 44 grammes of cement and 132 grammes of sand were mixed in a basin with from 12 to 15 cubic centimeters of water, depending upon the brand. That proportion of water which would give the strongest briquette when taken from the mould was used. The mixed materials were then compacted into the mould with a trip hammer. After one hundred and fifty blows the briquette was removed from the mould. In all cases the moulds were of the form recommended by the Committee of the American Society of Civil Engineers. All briquettes after mixing were kept in air for twenty-four hours and then immersed in water.

We had then forty briquettes of each kind immersed. Ten of each kind were broken after six days in water; ten more after thirteen days; ten more after twenty days, and the last ten after twenty-seven days in water. All neat briquettes were broken on a Riehle cement-testing machine, using an articulated, "parallel motion clip." All sand briquettes were broken on a Fairbanks machine, using the American Society clip.

The above-mentioned mechanical mixer or jig, the trip-hammer, and the parallel motion clip are described in an illustrated article in the *Engineering News* of January 3, 1891. The trip hammer is of the kind used in Germany for cement testing, and the sand briquettes were gauged in the German style (this is indicated in the table by the letters D. H.), with the exception that the American form of mould was used.

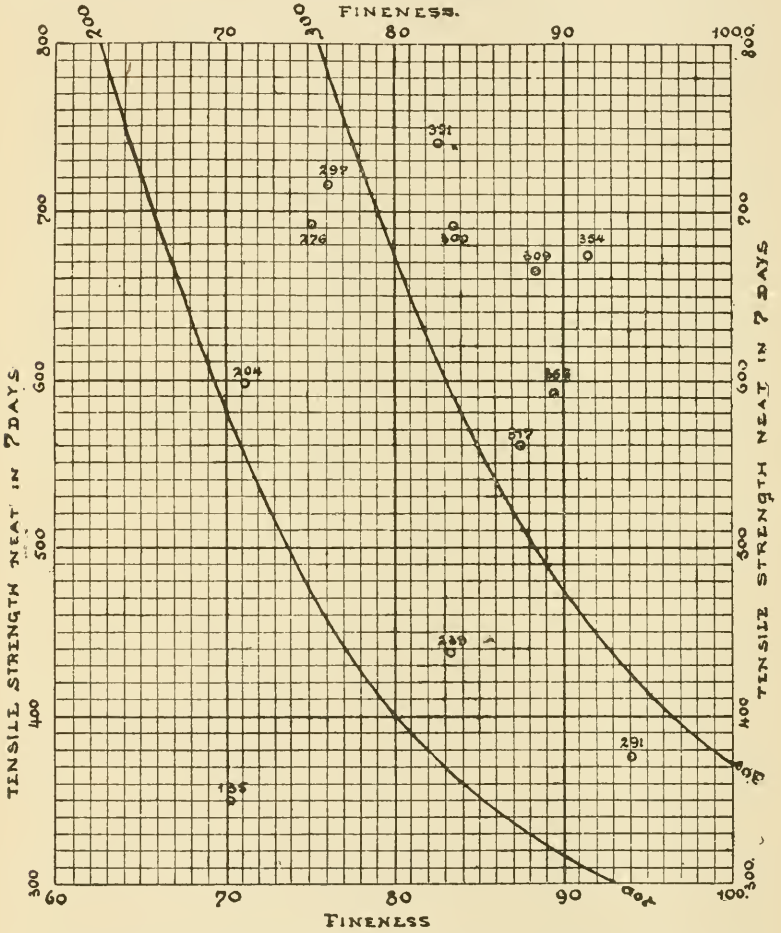
All of the tests were made by men of experience and skill in handling the apparatus and cement, and I am satisfied that each method had fair treatment. Of the twelve brands tested, five were German, two were English, three were American, one was Belgian and one was a German slag cement; all were Portland. After the tests were made the writer took advantage of the opportunity to compare the three ways of testing by their results.

One of the first requisites of a method of testing is uniform results with uniform material. To be thoroughly satisfactory the results should depend on the material only, and should, to a certain degree, be independent of persons, places, and seasons. To test a method by this standard would, however, take much time and study, and, moreover, concerted action by a number of experimenters. A good preliminary test of a new method can be made by observing the variation in the results of a given number of tests made at the same time with uniform material. In this case we have forty-eight groups of ten tests each to compare with forty-eight similar groups tested by another method. Taking the difference between the highest tensile breaking strength and the lowest in the group of ten tests we have the total variation for the group. Dividing this by the average of the ten tests, and we have the per cent. of variation for the group. With brand *A*, for example, we have in the first group of jig tests the highest test 728 pounds, the lowest 664 pounds, per square inch, making a variation of 64 pounds. The average of the ten tests was 691.2 pounds, so that we have a variation of 9 per cent. for the group.

Each number in columns 3, 4 and 5 of the table shows the per cent. of variation for a group of ten tests. A glance over these columns shows the comparative uniformity of the three methods. The forty-eight groups of briquettes mixed on the jig show an average variation of 14.7 per cent., and the maximum variation for one group is 33 per cent. The hand-mixed briquettes show an average variation of 22.5 per cent. and a maximum of 56 per cent., while the sand briquettes show an average of 27.3, and a maximum of 80 per cent. variation. We might say that the variations by the three methods are in the relation of 2, 3 and 4. This would indicate that the jig test is twice as reliable as the sand test, and 50 per cent. more reliable than the neat test by hand. It is proper to note that three sand briquettes of brand *K* broke in handling. These were not included in the averaging. No

other tests were thrown out. Brand *L* showed a tensile strength of 940 pounds per square inch in twenty-eight days by the jig test. This was the highest average of ten tests in the series. Ten sand briquettes of brand *E*, fourteen days old, showed an average breaking strength of 312 pounds per square inch, the best group of sand tests in the series.

Diagram, showing the relation between the strength of cements after seven days and their ultimate strength for different degrees of fine grinding.



The diagram above indicates to what extent we may trust a seven-day neat test combined with a sifting test to show the value of the

cement when used in mortar. After three months' time a mortar may be said to have practically reached its working strength. Ten sand briquettes (1 to 3 by weight) of each of the twelve brands were broken when ninety days old. Each cement was plotted on the diagram, using its neat tensile strength in seven days when mixed on the jig, and its fineness on a No. 100 standard sieve as co-ordinates. The location of a point shows the strength in seven days of a certain brand, also its fineness. The figures written at the point so found show the average of the ten ninety-day tests of the sample mixed with three parts of sand to one of cement. For example, brand *C* is 88.6 per cent. fine, and has a neat strength in seven days of 665 pounds per square inch. With sand it shows an average strength of 309 pounds in three months. In the diagram the best cements are found grouped together in the upper right hand corner above the line marked 300, while the poorest cement is

PER CENT. VARIATION IN A GROUP OF TEN TESTS FOR TENSILE STRENGTH.

BRAND.	Days old.	3 Neat. Jig.	4 Neat. Hand.	5 Sand. D. H.	BRAND.	3 Neat. Jig.	4 Neat. Hand.	5 Sand. D. H.
A	7	09	12	21	*G*	25	31	44
	14	09	17	60		11	35	22
	21	14	17	17		15	15	25
	28	30	08	32		27	12	14
B	7	09	18	20	*H*	10	56	31
	14	10	25	73		7	36	30
	21	10	20	21		20	36	19
	28	16	26	31		16	20	20
C	7	11	13	32	*I*	18	16	19
	14	10	16	18		15	27	29
	21	10	17	18		12	39	35
	28	27	08	10		33	36	28
D	7	10	14	14	*J*	17	37	22
	14	10	33	24		09	20	14
	21	9	18	11		24	30	32
	28	26	20	27		14	14	25
E	7	20	56	24	*K*	22	17	80
	14	10	20	22		21	22	23
	21	13	14	26		12	17	34
	28	17	25	17		11	19	34
F	7	09	09	20	*L*	09	17	25
	14	12	12	35		18	14	36
	21	09	17	11		10	36	33
	28	22	24	24		11	09	32
Average						14.7	22.5	27.3
Maximum						33	56	80
Minimum						7	8	10

found in the lower left hand corner below the line marked 200. As is generally conceded, the fineness plays an important part. The plotted points may be considered as points on a sloping surface, the figures showing the elevations. The lines marked 200 and 300 would then represent contour lines.

One of the disadvantages of the sand test that may be mentioned is that even when a standard sand is used, the heaviest cements do not stand a fair chance when the proportions are by weight. On a work the proportions are usually by volume. A standard sand for testing is often not to be conveniently obtained, and different sands have different specific gravities, so that on the whole the sand test by weight does not show the comparative value of cements as used in a work. Moreover, in diluting our cement with sand we practically reduce the size of our test. Who would advocate reducing the size of our briquette?

DISCUSSION.

D. J. Whittemore, Past-President Am. Soc. C. E.—In support of some of the deductions made by Mr. Russell, it may be that calculations based on extreme variations in tests of one group from those of another of a different or of a same brand of cement may be satisfactory, yet knowing that some cements are quite uniform as to quality and that others are not so, it appears to me that more conclusive evidence could be deduced from tests of one uniform quality and brand, in an attempt to judge of the merits of the various modes of gauging mortars. To secure this uniformity I have in many cases found it necessary to remix the cement in a dry state, and this can easily be accomplished by its repeated sifting into a funnel, whereby particles of cement from throughout the area of the sieve are brought together at the outlet of the funnel, thus insuring pretty thorough mixing. With cements thus made uniform, the method generally adopted by me in comparing different methods of gauging and testing has been, to ascertain the average variation of all tests from the mean of the individual tests of a group, and not that of the single tests of a group showing highest and lowest results.

In my paper in *Transactions*, Am. Soc. C. E., Vol. IX, page 329, upon Tensile Tests of Cements and an Appliance for more Accurate Determinations, it will be observed that I used the method suggested. Mr. Russell's testimony shows that the jig gauging is superior to that done by hand; and this fact, if it is a fact, leads me to again suggest the propriety of appointing a committee of our Society to consider and advise

PLATE LVIII
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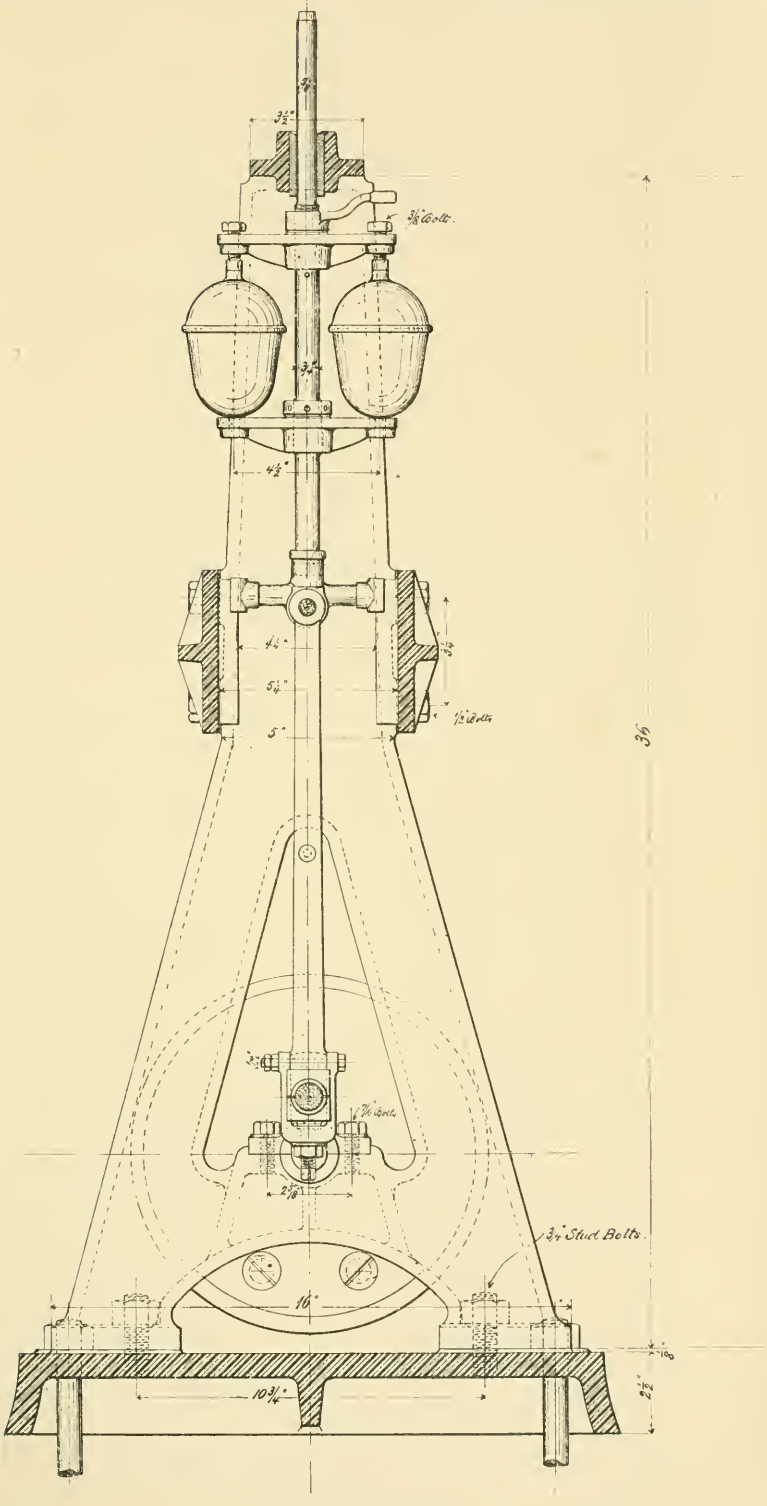


PLATE LIX
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. Nº 499.
RUSSELL ON CEMENTS, &c.

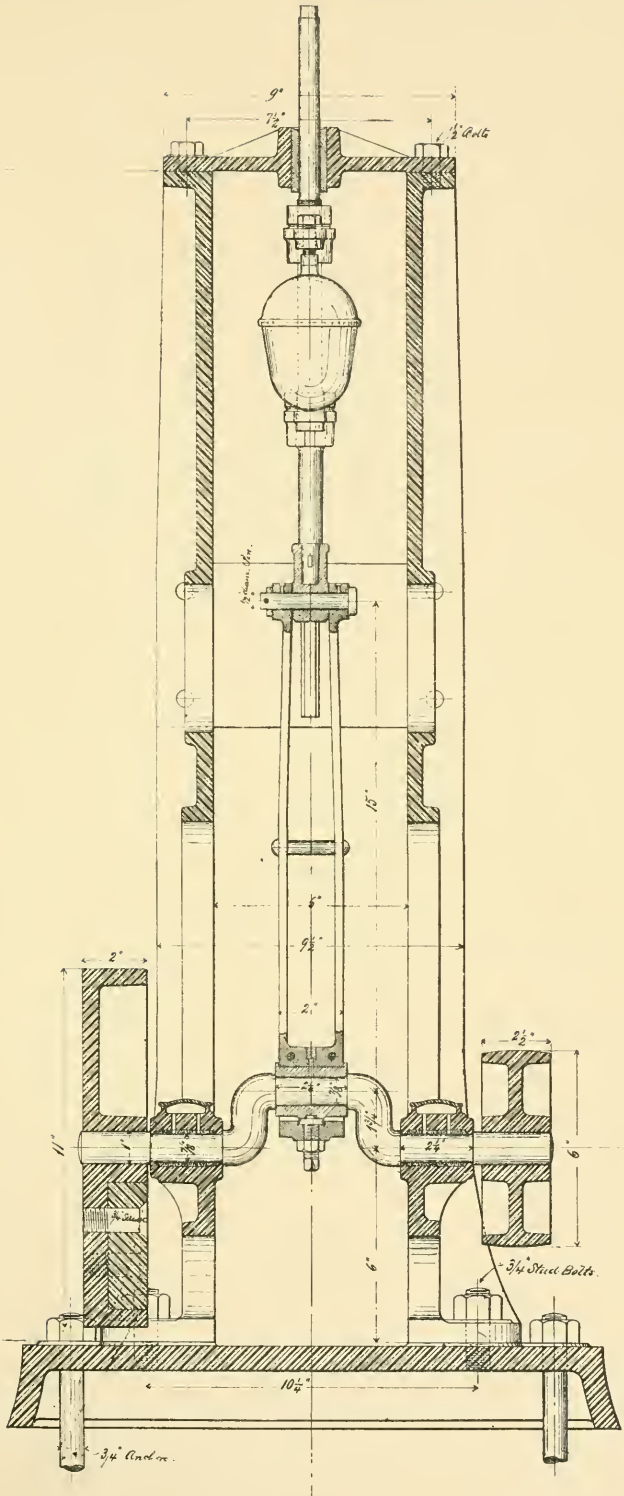
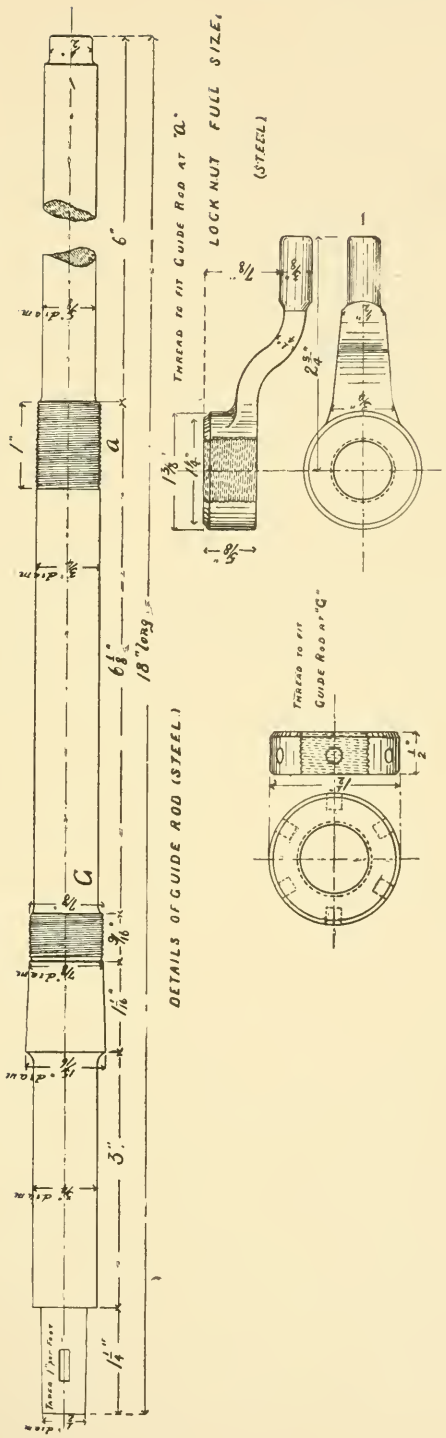
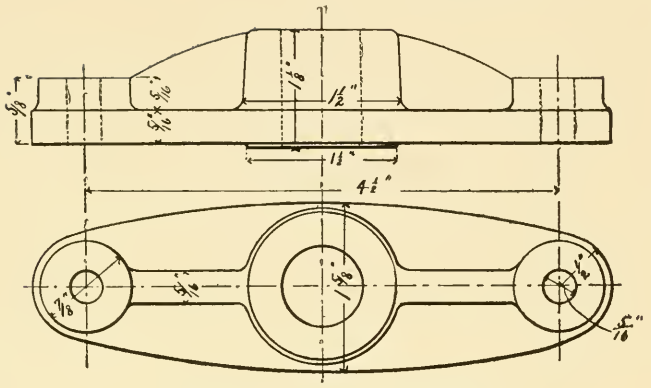
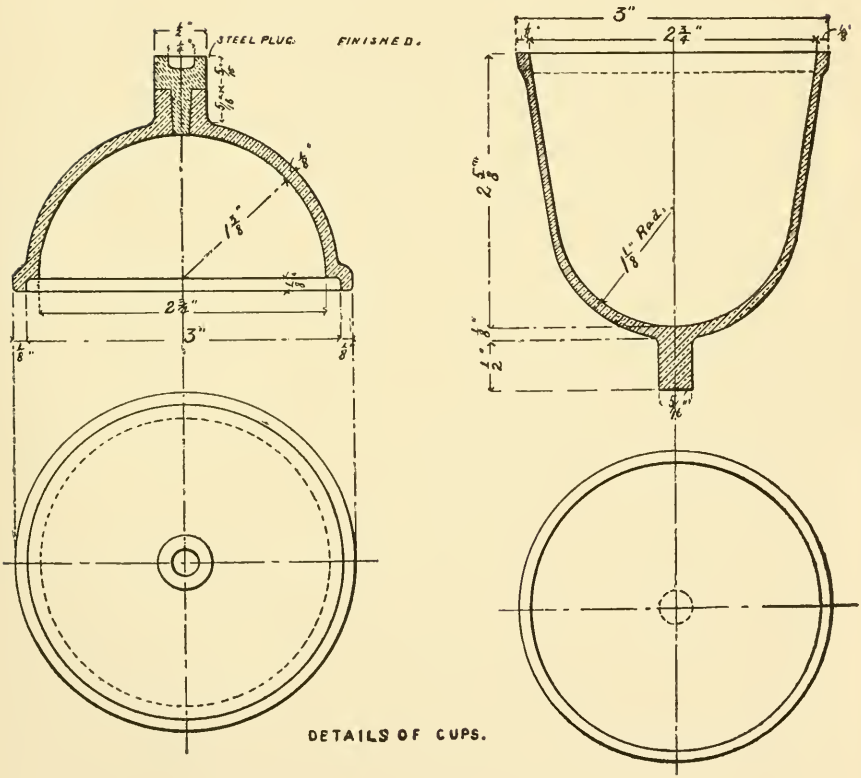


PLATE LX.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 499.
RUSSELL ON NEAT TESTS vs. SAND TESTS
OF CEMENTS.





DETAILS OF CUP SUPPORTS.



DETAILS OF CUPS.

additions to the rules heretofore formulated by the former Committee on Cement Tests. No one will dispute that those rules have been of great value to our members, but if they can be made more complete and definite, it is the evident duty of our members to make them so. It is my belief that the rules promulgated by our Society were at the time they were issued, and are now, sufficient to guide an intelligent experimenter towards the desired end; yet all experimenters are not remarkably intelligent, and it is to meet this emergency that I urge the adoption of more complete and definite rules, and the employment of some one or more of the numerous devices for fabricating briquettes that result in economy of labor and in securing uniformity.

It has been my lot to be interested in the manufacture of cements for many years, and, therefore, I have had more than usual opportunities to observe the work of novices in testing. It is not uncommon to find such whose first impressions of mortar for test purposes are, that it should have the consistency of a mud puddle. An instance of this character was recently brought to my notice: that of an engineer in an important city in Ohio, pretending to work under our rules, who used 42½ per cent. of water in gauging. I can understand how he blundered to the extent of say 12 per cent., but by what process of reasoning he tacked on the one-half per cent. is beyond my comprehension.

My experience has been that the manufacturers of cement have had but little reason to find fault with the work of the engineer who is expert in conducting tests, as they know that there is satisfactory foundation for any complaints he may make, but it is the work of the non-expert that gives them the greatest trouble, expense and discredit, and it is to meet such cases that I particularly urge a revision of our rules. Methods for better gauging and testing should be considered. Henry Faija, M. Inst. C. E., has devised a cement gauger now used by upwards of a hundred firms, corporations and persons in England, by which it is claimed that the cement can be reduced to the proper consistency as a mortar for forming briquettes, in considerably less time and with much less water than is possible by even the most expert hand gauging. He has also devised an apparatus for determining the soundness of cements in a few hours, which is based on the principle that moist heat accelerates the setting, and that if judiciously applied the indurating effects of several days under normal exposure of briquettes may be artificially given and determined in a few hours (Minutes of Proceedings, Inst. C. E., Vol. LXXV, pages 213–30).

Frederic Ransome, of London, has also devised a method of moulding briquettes in which the question of water to be used is entirely eliminated; simply by moulding the briquette with cement and any other admixture desired in a dry state, and allowing the same, while yet in the mould, to be submerged in the water. I understand that it is claimed that briquettes thus moulded show, when tested by groups, a variation from the mean of only about two and one-half per cent.

Our member, Charles D. Jameson, Professor of Engineering of the State University of Iowa, has devised a briquette making machine of which an illustrated description is given (in Plate LXII) by which two persons can fabricate six hundred samples per hour, and an examination of reported tests shows variations of about 12½ per cent. only in strength. All these appliances might well be considered, and the best recommended by a committee of the Society.

"The description of the machine is as follows: The main bulk of the machine is made up of the cylinder, *A*, which is flanged at the lower end, corresponding in shape and size to the upper part of the base seen in Fig. 5, and bolted to it by four bolts provided with fillers so as to separate the lower face of the cylinder and the upper face of the base exactly one inch. It is between these faces that the moulding plate, *L*, seen in Fig. 4, swings, the fillers on the bolts acting as stops. The cylinder is made up of two parts bolted together. The bore is of the shape and size of a briquette and may be seen in plan in Fig. 3. In this bore works a solid plunger of such a length as to cover the feeding hole, *M*, when at its lowest point, and connected to the lever, *B*, by the connecting rod, *C*. The fulcrum of *B* is supported by *D*. About the point, *E*, Figs. 3, 4 and 5, the plate, *L*, swings in such a way that when at either extreme one of the openings is directly beneath the cylinder, while the other is directly over the plunger, *F*, the top of which, when at its lowest point, is a little below the under face of the moulding plate. These plungers, *F F*, are of the shape of the opening in the moulding plate and are raised by the levers, *G G*, and are to force the briquettes from the mould. They are each provided with two guide pins, *K K*, which work in holes in the base plate. The levers, *G G*, are attached to the base plate by bolts seen in Figs. 3 and 5. Small pieces, *H H*, are riveted to the levers and pass under the plungers. It is by means of these that these plungers are lifted. At the outside of the cylinder is the hopper, *I*, seen only in Fig. 1, which communicates with the interior through the opening *M*, seen in section in Fig. 6.

"The method of operating is as follows: The lever, *B*, being raised so that the lower end of the piston or main plunger is above the hole in the side of the cylinder communicating with the hopper, cement or mortar is put in the hopper and pushed down into the cylinder. The moulding plate is pushed against one of the stops, so that one of the openings is beneath the bore of the cylinder. The long lever, *B*, is forced down, causing the plunger to force the cement into the opening in the moulding plate. After this, the moulding plate is swung against the other stop, cutting off the briquette, placing it over the plunger, *F*, throwing the other opening in the moulding plate directly beneath the cylinder. The lever, *G*, is lifted, raising the plunger, *F*, and forces the briquette out of the mould, after which it is removed. The plunger, *F*, is then pressed down, the main lever, *B*, pressed down, the moulding plate swung back to the first position, the other plunger lever, *G*, lifted, and another briquette is ready to be taken away, and so on. After making two or three briquettes, the main lever is lifted and more material put into the cylinder. The machine is best operated by two men, one to feed and operate the long lever, and the other to swing the moulding plate, remove the briquettes and lower the plungers, *F F*. The pressure on the briquette is about 175 pounds per square inch." *

* Copied from *The Transit* of December, 1890.

PLATE LXII.
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXV. Nº 499
WHITTEMORE ON
NEAT TESTS VS. SAND TESTS OF CEMENTS.

FIG. 1

FIG. 2

FIG. 3

FIG. 4

FIG. 6

FIG. 5

I beg Mr. Russell's pardon for wandering so far from the particular feature of his paper, in my discussion of it, and offer as my excuse that the subject is far reaching in its nature, and that what is suggested by me is certainly germane to the subject of cement tests in general.

W. W. Maclay, M. Am. Soc. C. E.—The paper by Mr. Russell is a very interesting one, but I do not agree with his conclusions as to the reliability and comparative value of the three methods of testing experimented upon; namely, that the jig test is twice as reliable as the sand test, and 50 per cent. more reliable than the neat test by hand.

In the first place, it seems to me the comparison should have been confined to the neat test by hand with the neat test by machinery or jig, and to do this it would have been better to have taken the averages of the ten briquettes in each set in columns 3 and 4 of the table, instead of the percentage variation between the extreme breakings; then with the same material to have made up duplicate or triplicate sets, and have compared the resulting averages with one another. I think such a table would give one a better idea of the accuracy than could be obtained from each system. For example, while it is recognized in the German rules for testing cement that in cases of dispute machinery shall be resorted to, and Dr. Boehme's apparatus be used both for mixing and compacting the material in the moulds, yet in my experience with a person trained and skillful in making briquettes, I have obtained average results from re-tests of the same cement made by hand, both neat and mixed with sand, not differing from the original tests more than 2 to 5 per cent., and this not as the exception but as the rule. Such hand-mixing seems to me about as accurate as it is possible to make cement tests by any method.

After neat tests by hand and machinery had been prepared the sand tests with duplicate and triplicate sets could be made, and then the average results compared with each other, but not with the neat tests; for while the sand tests bear some relation to the neat tests, they do not vary in the same ratio, even when considering the element of fineness. The value of the sand test is based on the fact that it is a much nearer approach to the actual condition under which cement is used in construction than the neat test.

Dr. Michaelis says: "If the question were raised as to testing cement, neat or mixed with sand, and if it were acknowledged that only one of these two methods were permissible, then there was no doubt that the decision must be in favor of the sand mixture."

Mr. Russell says: "After three months' time a mortar may be said to have practically reached its working strength." From experiments carried on during the last fifteen years on many brands of cement, there occur only a few exceptions in which the sand briquettes have not increased steadily in strength up to three years.

W. R. Hutton, M. Am. Soc. C. E.—Upon Mr. Russell's paper it may

be remarked, that the method of filling moulds with the trip hammer develops the very highest qualities of the cement and is deceptive if results are compared with those obtained in other ways. It is eminently a manufacturers' test. Pressure by means of weights applied according to some fixed standard would perhaps better replace the uncertain pressure of the trowel in the hands of the experimenter.

Would it be more fair to refer to the percentage of variation to the mean results instead of the opposite extreme ?

S. Bent Russell, M. Am. Soc. C. E.—In reply to questions that have been asked, I will attempt to describe a little more fully the method used in gauging the sand briquettes for the experiments.

Dr. Boehme's hammer machine consists of a trip hammer operated by a hand-crank, a mould, and a plunger which fits the mould. The trip hammer is provided with an automatic counting gear which stops the movement after 150 blows. The hammer strikes the plunger, which passes through an iron block which rests on the briquette mould and conforms to it in outline. In using this apparatus, the cement, sand and water are mixed in a hand-basin with a spoon until the mixture is apparently uniform. It is then of a mealy consistency. Sufficient of this mixture to form one briquette is then poured into the mould and plunger block, the plunger set on and the hammer started. The crank is turned as fast as the fall of the hammer will permit, which is somewhat deliberately, until it stops automatically. The plunger and cap are removed and the briquette is then shaved off with a knife and at once removed from the mould. If too much or too little water is used the briquettes will not come out well.

The letters D. H. in the table signify that the above-described method of gauging has been used, and are an abbreviation of "Dutch Hammer," which is the laboratory name of the Boehme machine.

When the jig was devised, the intention was to obtain a machine which would imitate as closely as possible the gauging done by hand. The mortar is mixed in the jig of about the same consistency as in the hands of a skilled experimenter, and the results obtained agree closely with those of the most skillful and painstaking hand mixing. The adoption of such mechanical methods seems to me to be a step in the right direction.

I am much pleased to hear from our Past-President, who was a member of the Society's committee on uniform tests of cement, that he is in favor of a revision of the committee's report, made nearly seven years ago. I am heartily in favor of such a revision, and have already advocated it elsewhere. After seven years' use we should be able to criticise the report intelligently. For myself, I should say that if there be faults in the report they are rather sins of omission than of commission.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—September, 1891.)

THE MARENT GULCH VIADUCT.

By GEORGE S. MORISON, M. Am. Soc. C. E.

The Portage Viaduct was built in the spring and early summer of 1875. The Marent Gulch Viaduct was built in the fall of 1884 and the following winter. The height of the iron work at the highest tower is practically the same for the two viaducts, and the length of the viaducts is also very nearly the same. The Portage Viaduct was built in the hurry of a pressing necessity, with little time to investigate relative economies or to prepare well considered plans. No such haste existed in the case of the Marent Gulch Viaduct, and the plans were carefully considered and worked out. The Marent Gulch Viaduct, therefore, represents ten years' advance in iron viaduct construction, besides such improvements as careful preparation always gives opportunity for.

The Marent Gulch Viaduct is in Montana, on the main line of the Northern Pacific Railroad, a little west of Missoula. Like the Portage Viaduct, it replaced a wooden viaduct already built on the same location, but unlike the Portage Viaduct, the replacement was made, not after destruction of the wooden viaduct by fire, but in anticipation of such destruction, which, in the dry climate of Montana, was considered a risk which the traffic of a transcontinental railroad should not be exposed to.

The wooden viaduct was built only two years before the iron viaduct. It was entirely of wood, without masonry foundations. It was built in alternate spans of 20 feet and 50 feet, two bents being braced together under the short spans, so as to form timber piers. The 50-foot spans were Howe trusses with wooden angle-blocks, which deflected so much that it was found necessary to reinforce them before the completion of the iron viaduct.*

The dimensions of the new viaduct were fixed in part by those of the old viaduct. The distance from center to center of piers in the new viaduct was made 140 feet, which brought the piers under the centers of the alternate long spans of the old wooden viaduct. The distance between the centers of the trusses, or the width of the piers at the top, was made 20 feet, this being the same as at Portage, but the object of adopting this width was to raise the new iron trusses without disturbing the old wooden ones (an object which the necessary renewal of the new trusses partially overcame), whereas the width of the Portage Viaduct was fixed with reference to substituting a double track superstructure for the original single track superstructure (which has never been done).

The general arrangement of the Marent Gulch Viaduct is shown on Plate LXIII. It consists of five spans, each 116 feet 8 inches long; of four towers, each measuring 23 feet 4 inches on top, and four girder spans of 30 feet each, making the total length of the entire structure from outside to outside of iron work 796 feet 8 inches. The total height from top of masonry to top of stringer is 201 feet 9 inches, this being at the high side of the two central towers. The viaduct is straight, but is built on a grade of 1.962 per cent.

Substructure.

The substructure consists of two small abutments and twenty-four small piers. The details of these piers are given on Plate LXIV. They are generally founded upon rock, but in two instances they were founded on clay and gravel, the area of the foundations being increased; and in two other instances they were founded on piles driven in clay and gravel and reaching a hard material which probably was rock.

Two plans were prepared for these piers; one was for masonry piers,

* A historical description of the Marent Gulch Viaduct, including the timber structure, will be found in a paper by S. D. Mason, read before the Civil Engineers' Society of St. Paul, and printed in the Journal of the Association of Engineering Societies for April, 1889.

which was adopted; the other consisted of concrete piers inclosed in wrought-iron shells covered with cast-iron caps. The author's judgment was in favor of the concrete piers, as he believed they could be more cheaply constructed. General Adna Anderson, M. Am. Soc. C. E., who was at that time Chief Engineer on the Northern Pacific Railroad, however, preferred the masonry piers, which were therefore built. They were built of limestone from the quarries at Kasota and Mankato, Minnesota, the stone being carried more than 1 300 miles by railroad. The abutments are granite. Each small pier had a 2-inch anchor rod built into the masonry, extending from the concrete or bottom of the foundation to the top, thus tying the whole masonry together.

The quantities of material in the substructure are given in the following table:

Pier or Abutment.	Excavation.		Concrete. Cubic Yards.	Cut Stone Masonry. Cubic Yards.
	Rock. Cubic Yards.	Earth. Cubic Yards.		
Earth Abutment	36	.....	.....	23.12
Pier A	48	.....	.....	6.47
B	48	.....	3.71	6.47
C	39	.....	8.63	22.91
D	66	.....	.....	37.01
E	66	112	15.67	16.01
F	30	143	28.18	38.03
G	56	105	15.26	28.74
H	98	389	5.85	55.76
I	132	229	.....	21.77
J	125	116	21.70	33.30
K	77	470	41.92	31.20
L	.....	280	67.60	37.58
M	.....	240	56.46	37.58
N	.....	195	63.16	37.16
O	.....	389	122.38	27.57
P	122	702	48.89	27.38
Q	93	165	31.48	60.53
R	241	30	.....	51.78
S	58	124	.....	32.82
T	119	.....	.....	10.17
U	18	.....	.....	14.57
V	15	.....	.....	13.15
W	18	.....	.....	5.93
X	18	.....	.....	5.93
West Abutment	122	.....	.....	40.39
	1 645	3 689	543.80	722.33

Superstructure.

The superstructure comprises two high towers, two towers of medium height, four bents, five pin-connected deck spans and four plate girder spans. Except for the slight irregularity caused by the grade, the structure is perfectly symmetrical, an unusual thing in a viaduct of this class.

The towers were made of the same length on top as the panels in the spans between them, so that the floor system is entirely uniform over the trusses and the towers, the length of towers being 23 feet 4 inches. The towers are built with a batter of one in forty-eight in the direction of the bridge, which increased the length of the high towers at the base to 31 feet $2\frac{1}{16}$ inches, and with a side batter of one in six (the Portage batter is one in eight), which makes the width of the high towers at the base 83 feet $2\frac{1}{4}$ inches.

The strain diagrams of the towers are given on Plate LXV. They were calculated to resist four classes of strain:

First.—The live load brought on top of the tower.

Second.—The dead load brought on the top and also at intermediate points.

Third.—The wind pressure brought on the top and also at intermediate points.

Fourth.—The traction strain due to the pull of a locomotive or the resistance of brakes, all of which is applied at the top of the tower.

An examination of the diagram shows that with a maximum wind pressure and no live load—an inconceivable condition—a small tension may exist at the bottom of the tower on the windward side. It also appears that with a combination of all maximum conditions, tension may exist from the effects of traction at the same point in one leg only of the four legs of a tower; this is a very improbable supposition, and the existence of tension at only one of the four points of support could not endanger equilibrium.

The width of the towers at the base is so great that it became necessary to make some provision for expansion and contraction in the horizontal members. The arrangement adopted is shown on Plate LXVIII. On one side the foot of the post is bolted securely to the cast wall plate which is anchored to the masonry. On the other side the attachment is made through a rocker of cast-steel, which rocker permits of expansion and at the same time is able to resist some tension. The superstructure

is bolted rigidly to the tops of the towers, excepting at two points on towers *C* and *F*, where expansion rollers are provided, these rollers being of steel running on steel castings. The expansion, therefore, of the central span is taken up by the spring of the lofty towers which support it.

The full details of the towers are given on Plates LXV to LXXI inclusive.

It will be observed that each tower rests on four points only, the stiffening members (which in the Portage Viaduct rest on the old stone piers) being sustained by the diagonal bracing.

The spans consist of deck pin-connected trusses, the full details of which are given on Plates LXXII and LXXIII. It will be observed that there is a complete iron floor, the floor-beams resting on top of the cords and the stringers on top of the floor-beams, the stringers being continuous and uninterrupted. On these stringers is laid the wooden floor, the details of which appear on Plates LXVII and LXXII.

The actual weights of material in the superstructure are as follows:

	Pounds.	Pounds.
Two high towers, *D* and *E*	549 113	
Two towers, *C* and *F*	276 804	
Bents, *B* and *G*	34 036	
Bents, *A* and *H*	8 398	
Total towers and bents		868 351
Five deck spans	466 700*	
Floor system	297 827	
Total deck spans	764 527	
Four plate girder spans	40 161	
Total spans and floor		804 688
Total iron and steel		1 673 039

The superstructure was manufactured from designs prepared by the author, and except 160 eye-bars, was made by the Union Bridge Company at their Buffalo shop; the 160 eye-bars were made by the Passaic Rolling Mill Company at Paterson, N. J. The superstructure was inspected by

*In this table the weight of the eye-bars is taken at 141 660 pounds, which is the actual weight. The original estimate was 118 938 pounds, but the section of the bars was increased 20 per cent. beyond the original provisions, the first lot of bars furnished not meeting the requirements of the specifications.

Mr. George A. Lederle, Assistant Engineer, who, on the completion of the work at the shops, superintended the erection in the field. All work at the bridge site was done by the company's own men without a contract.

The stone piers were built and the towers erected before any change was made in the timber viaduct, the timber structure serving as a scaffold. The stone work was handled by derricks placed on the hillside, and the material was passed from one derrick to the other until set in its final position. The iron work was handled by a derrick car worked from the top of the bridge, and the sections lowered into place directly from the car. The derrick car had two booms (see Plate LXXIV), and the members were picked up by this car from a yard east of the bridge; the car was run out on the bridge with the two members swinging from it and balancing each other; the members were then lowered into position, none of the principal members being handled on push-cars or moved forward in any other position except when swinging from the derrick-car. In this way the towers were raised to the top.

The trusses would also have been raised without disturbing the timber work, except for the fact that the old Howe trusses had been supplemented by new trusses placed on the outside of them after the erection of the iron bridge had been determined upon and before it was completed; it was therefore necessary to move the timber trusses toward each other, so as to give room to raise the iron trusses. The material for the new iron trusses was handled in the same way as that for the towers, and erected on staging supported by the timber towers and by hangers from the wooden spans.

After all the new trusses were swung the new floor system was put on, one new span being handled at a time. This change included the removal of the two timber spans, the placing of six iron floor-beams, of seven spans of iron stringers, and of the ties of the timber floor, all of which was accomplished in from six to seven hours and between the passage of trains. After the completion of the new iron structure, it was in turn used as a staging to take down the timber structure; this was handled with the same derrick-car, the timber being raised to the floor, loaded on a flat car and carried away for use elsewhere.

The work at the bridge site was not driven. Work was begun there about the first of September, 1884; the new iron structure was completed in April of the following year, and the wooden structure had been entirely removed a month later.

The cost of the structure was materially increased over what the cost of a similar structure in other parts of the country would have been, by three items: first, the fact that much of the work was done in the winter; second, the high rate of wages in Montana; third, the large amount of transportation.

The cost of the viaduct complete was as follows:

Foundations	$21 664 59		
Masonry	30 079 81		
			$51 744 40
TOTAL PIERS.			
Iron work for towers and bents	$33 431 51		
Erection towers and bents	15 756 93		
		$49 188 44	
TOTAL TOWERS AND BENTS.			
Iron work of spans and floor	$30 065 73		
Erection of " "	6 528 21		
		36 593 94	
			85 782 38
TOTAL IRON WORK.			
Timber floor and track			4 923 49
Painting			1 826 74
Engineering and incidentals			9 085 15
Total			$153 362 16

To this was added the charge for transportation on the line of the Northern Pacific Railroad, which made the entire cost of the structure $178 105 64. If we compare this cost with that of the Portage Viaduct, only the iron work, floor and painting should be included, these items aggregating $87 609 12. The structure itself, without the masonry foundations, could have been erected in the eastern part of the country for about $75 000.

The total area circumscribed by a polygon embraced between the top of the stringers and a broken line connecting the centers of the bridge seats on the several pieces of masonry is 95 735.52 square feet. The total area of a broken line embraced between a line forming the bottom of the stringers and the same broken line is 93 712.98 square feet. The iron work included within the larger polygon is 1 673 039 pounds, being 17.47 pounds per square foot, and within the smaller polygon is 1 461 879 pounds, being 15.59 pounds per square foot.

If we consider only the portion of the bridge where it is highest, the total area included in a quadrilateral the top of which is the top of the stringers, the bottom of which is a line connecting the centers of towers

D and *E* at the level of the top of the masonry, and the sides of which are the axes of the two respective towers, is 28 189 square feet; the weight of iron within this area is 421 026 pounds, or 14.93 pounds per square foot. If we take off the height of the stringers the area becomes 27 833 square feet and the weight 403 119 pounds, or 14.48 pounds per square foot. If we also take off the height of the floor-beams, the area becomes 27 367 square feet, the weight 373 658 pounds, and the weight per square foot 13.65 pounds. If we take off the floor-beams, but retain the stringers, which practically corresponds to the conditions at Portage, where the cross timber floor-beams rest on the cords, the area becomes 27 723 square feet, the weight 391 565 pounds, and the weight per square foot 14.12 pounds. These several figures are interesting for purposes of comparison.

The appearance of the structure is good, and if the author had occasion to build a bridge on the same location, he would make comparatively few changes, one of these changes being, perhaps, the substitution of a stiff riveted bracing for the adjustable bracings of the towers.

Plates LXXV and LXXVI are views during the erection and after the completion of the viaduct.

PLATE LXIII
TRANS.AM.SOC.CIV.ENG'RS
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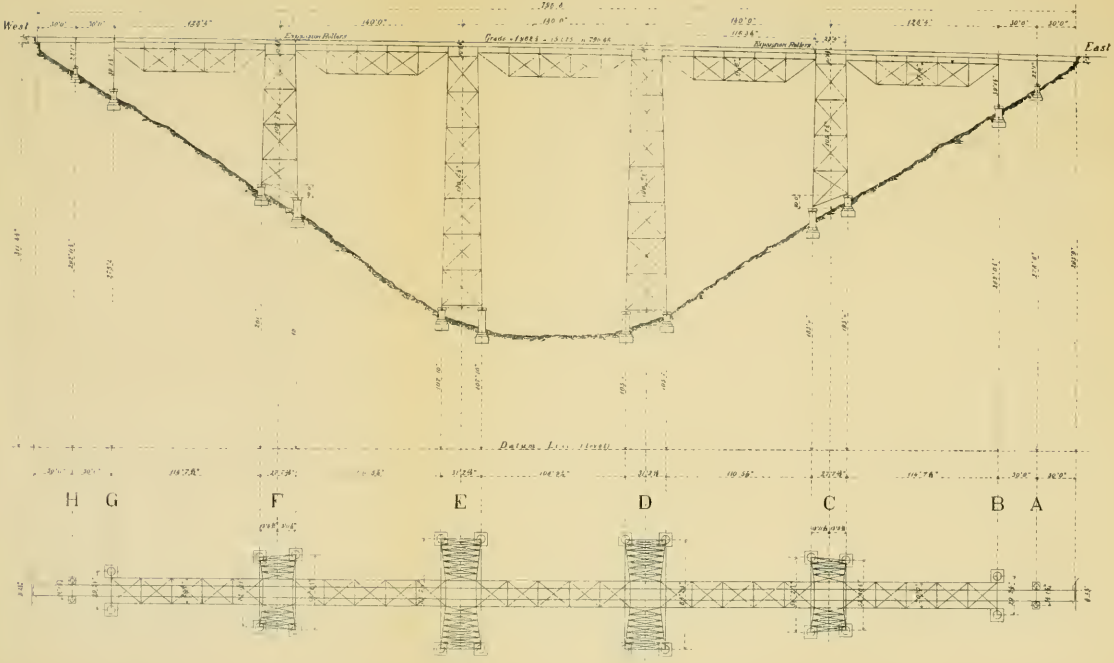
MORISON ON
THE MARENT GULCH VIADUCT.

N. P. R. R.

MARENT VIADUCT

General Elevation and Plan.

Scale



N. P. R. R.

MARENT VIADUCT.

MASONRY and FOUNDATIONS

PLATE LXIV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. N° 500.
MORISON ON
THE MARENT GULCH VIADUCT.

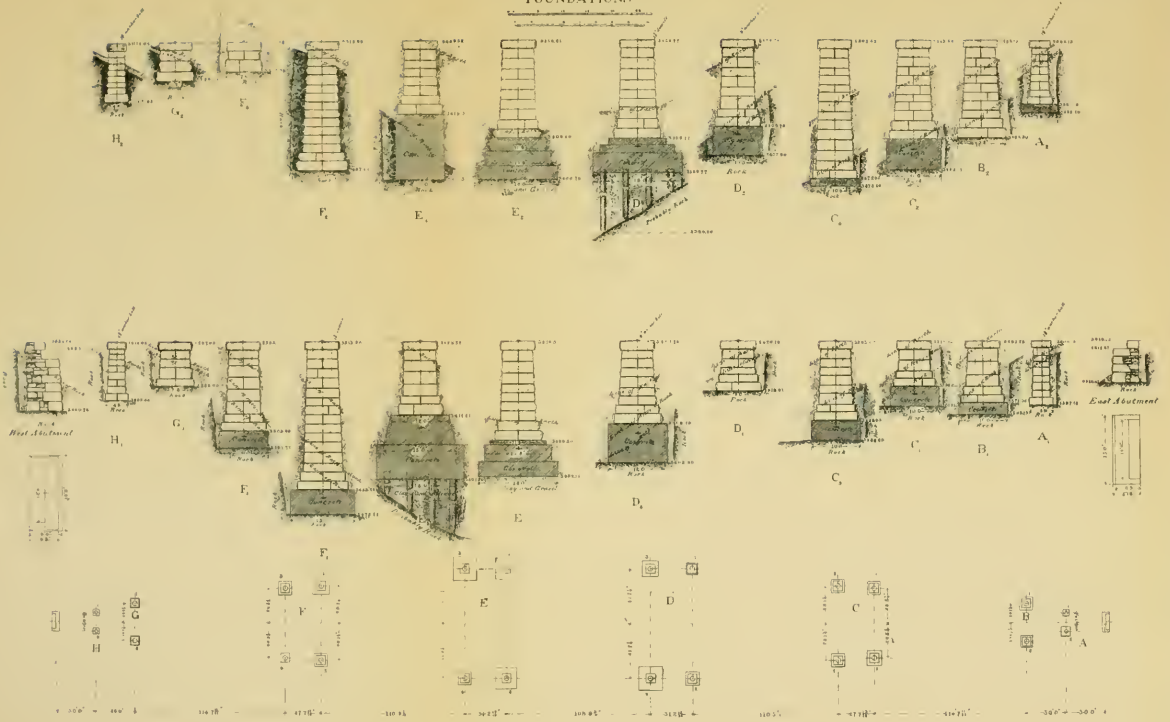


PLATE LXV
TRANS.AM.SOC.CIV.ENG'RS.
VOL XXV N°500
MORISON ON
THE MARENT GULCH VIADUCT

N P R R

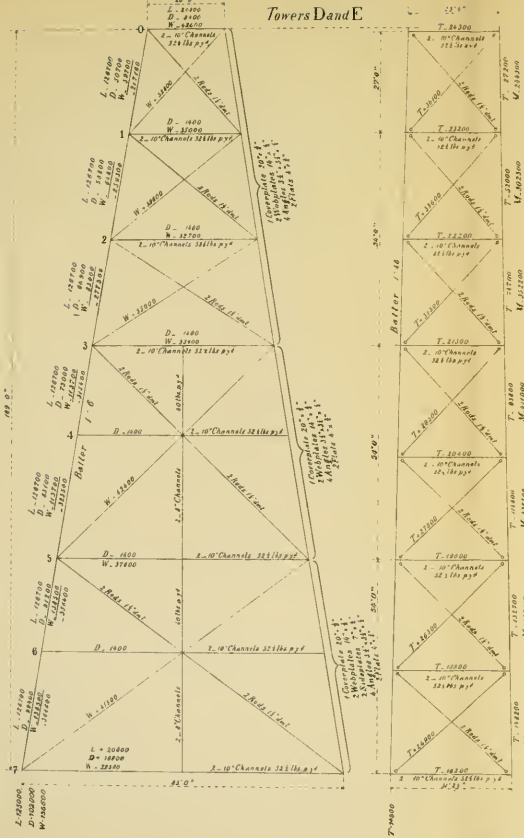
MARENT VIADUCT

Scale

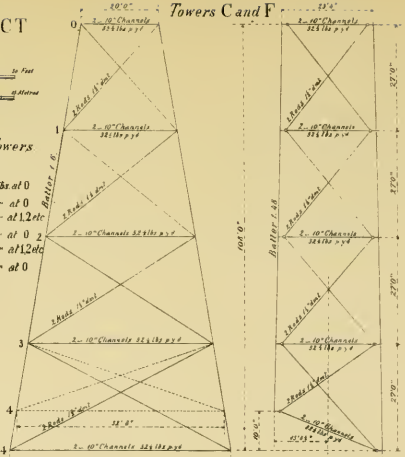
0 5 10 20 30 40 50 Feet

0 1 2 3 4 5 10 15 Metres

Towers D and E



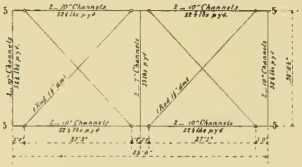
Towers C and F



Strains and Sections of Towers

L = Strains due to Live Load of 125000 lbs at 0
D = " " " Dead " at 30000 " at 0
and 6000 " at 12 etc
W = " " " Wind Pressure of 42000 " at 0
and 6000 " at 12 etc
T = " " " Traction of 24500 " at 0
M = " " " L.D.W.ST.

Central Section



Plan at 5

Details of Plate Girders and Bents.

N. P. R. R.

MARENT VIADUCT

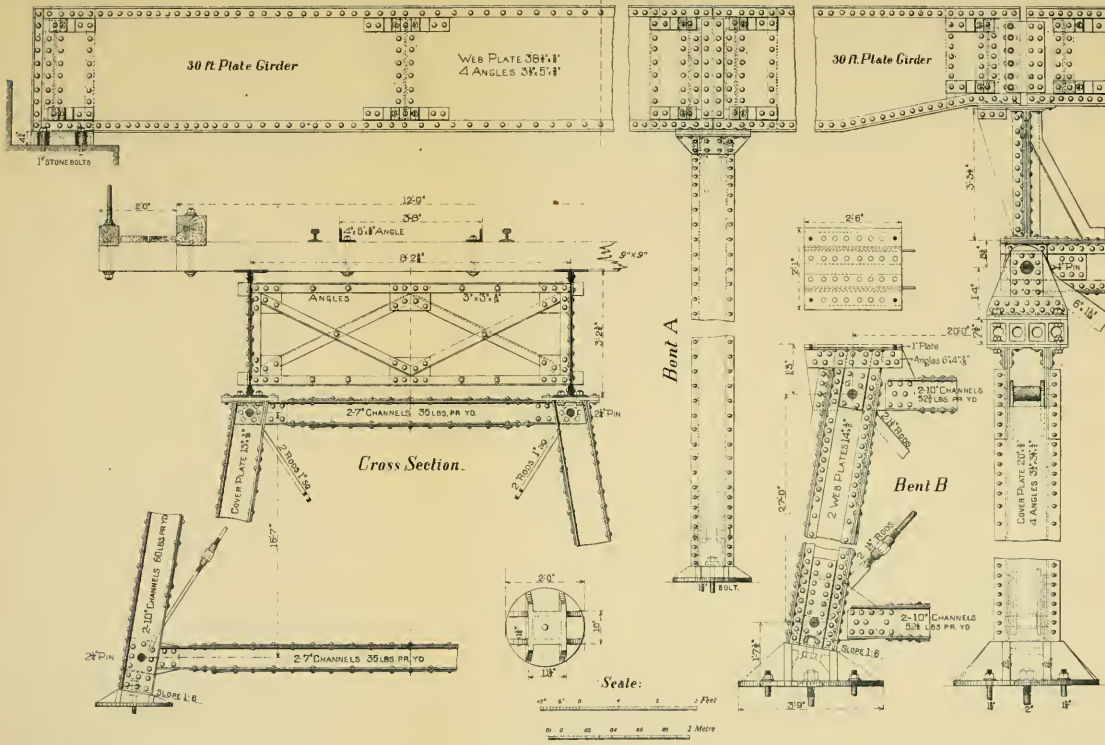


PLATE LXVII
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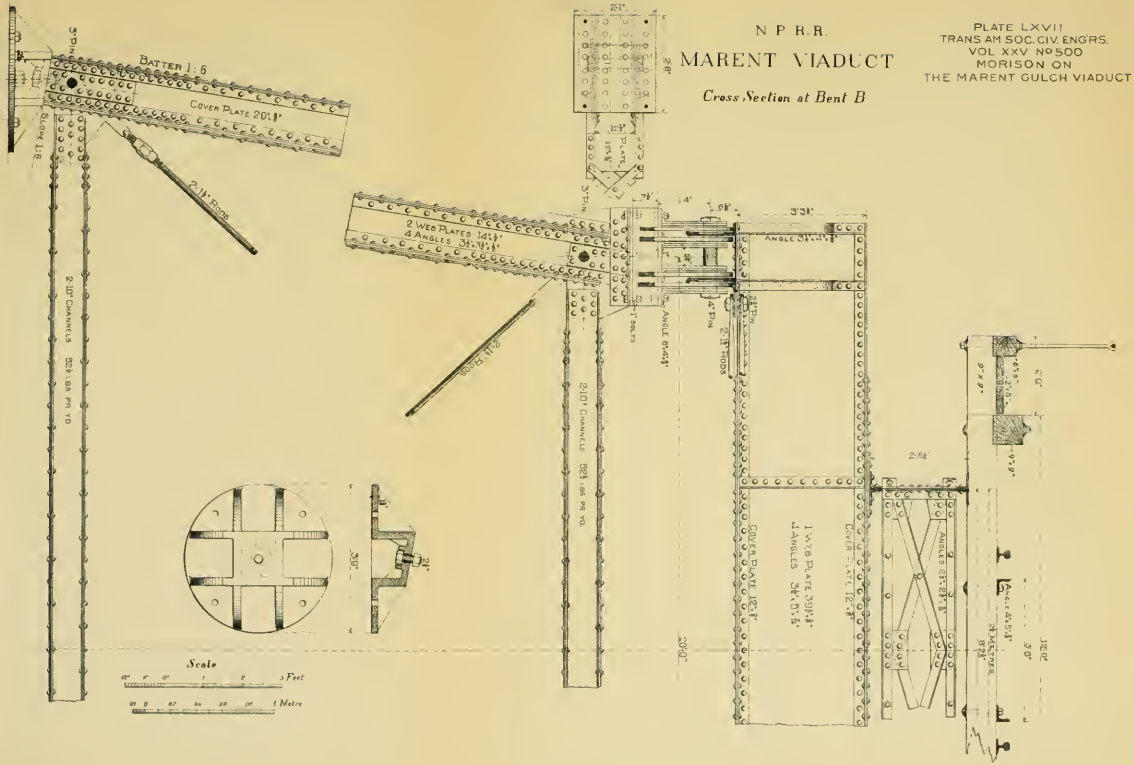
MORISON ON
THE MARENT GULCH VIADUCT

N P R.R.

MARENT VIADUCT

Cross Section at Bent B



Tower C & F

N. P. R. R

MARENT VIADUCT

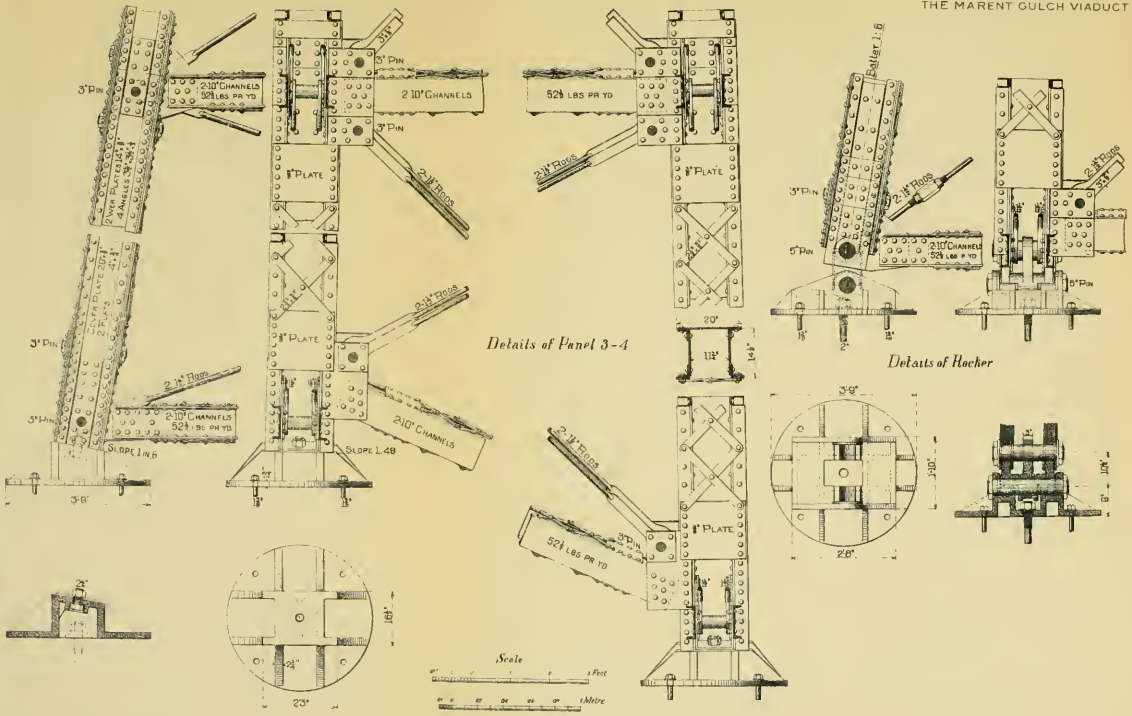


PLATE LXIX
TRANS. AM. SOC. CIV. ENGRS.
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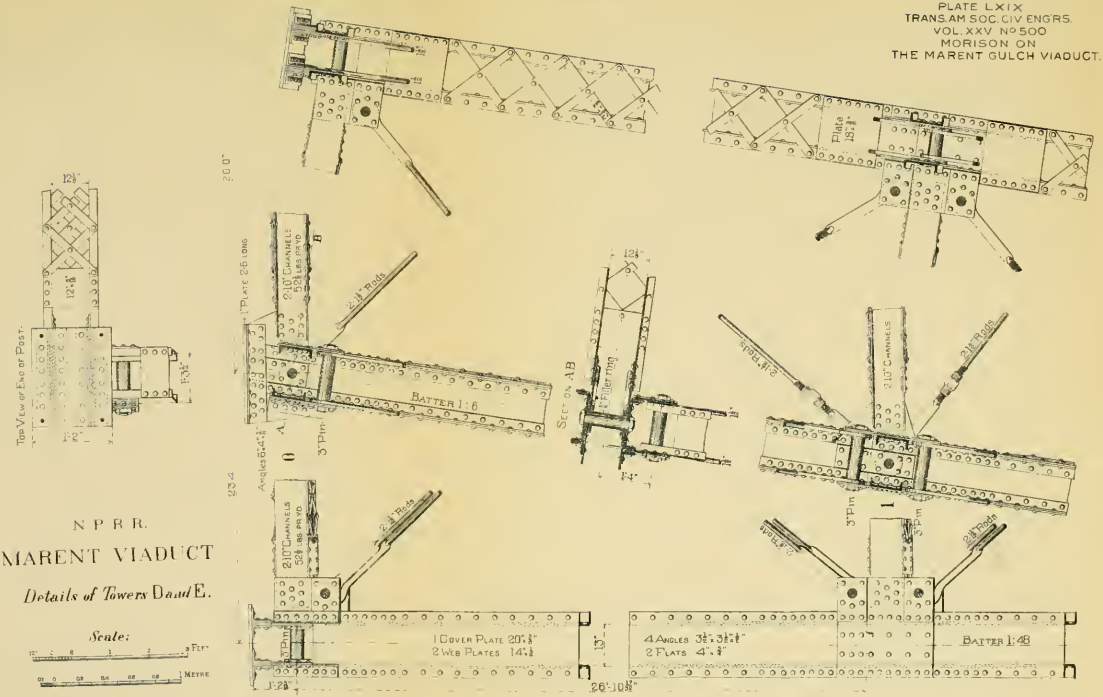
MORISON ON
THE MARENT GULCH VIADUCT.

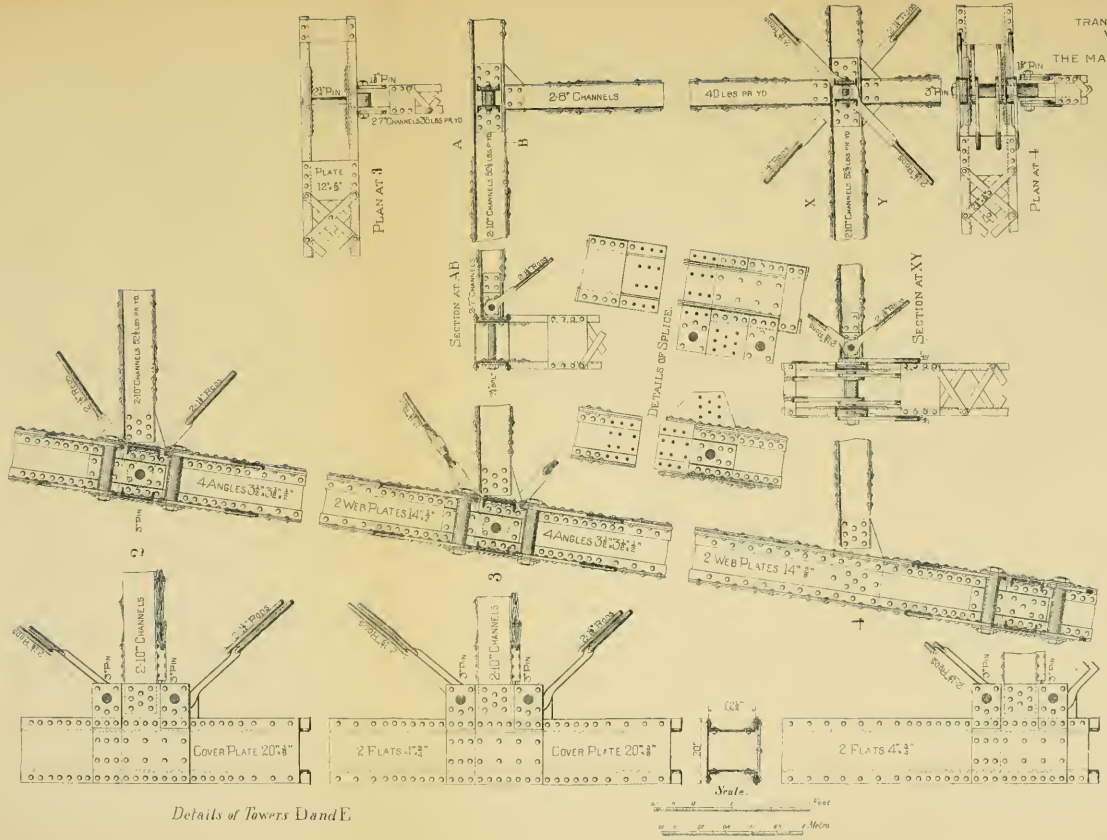


N. P. R. R.

MARENT VIADUCT

Details of Towers D and E.

Scale:



Details of Towers D and E.

PLATE LXXI
TRANS. AM. SOC. CIV. ENG'RS
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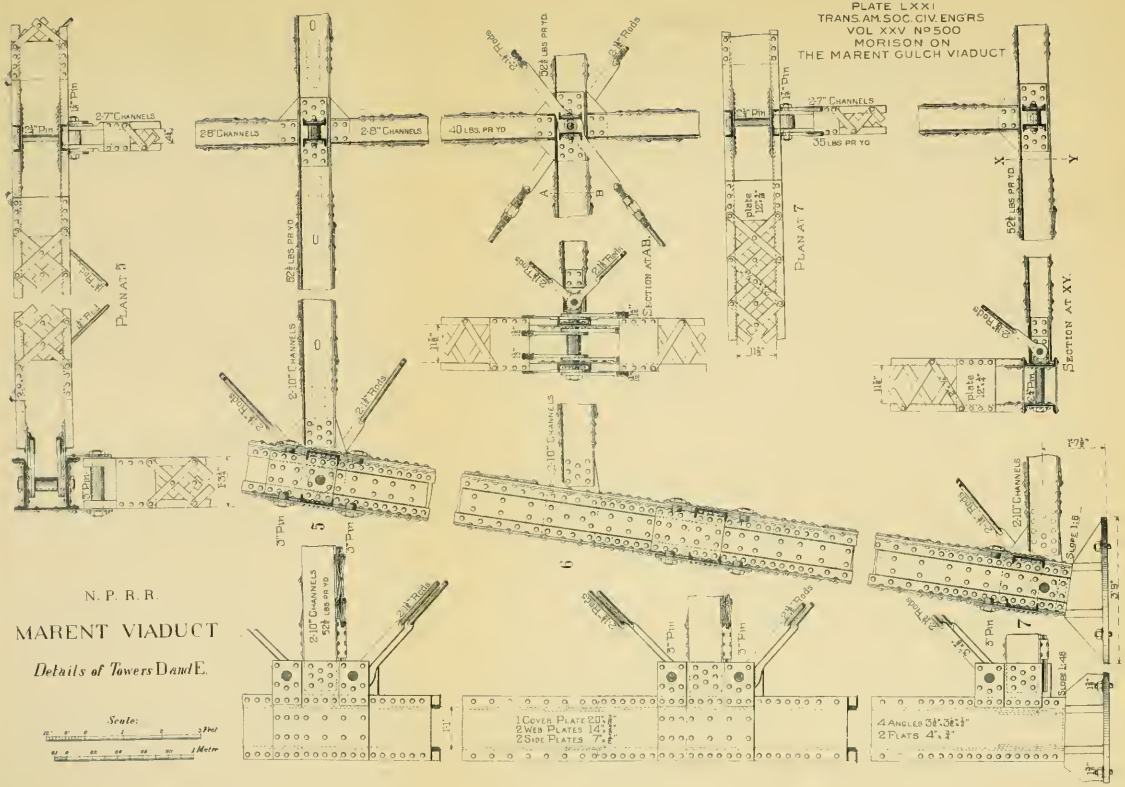
MORISON ON
THE MARENT GULCH VIADUCT



N. P. R. R.

MARENT VIADUCT

Details of Towers D and E.

Scale:
3 Feet
1 Metre

N P R R

MARENT VIADUCT

Scale

0 1 2 3 4 5 10 15 20 Feet

0 1 2 3 4 5 6 Metres

PLATE LXXII
TRANS AM SOC. CIV ENG'RS.
VOL XXV No 500
MORISON ON
THE MARENT GULCH VIADUCT

General Elevation and Plan of Deck Span

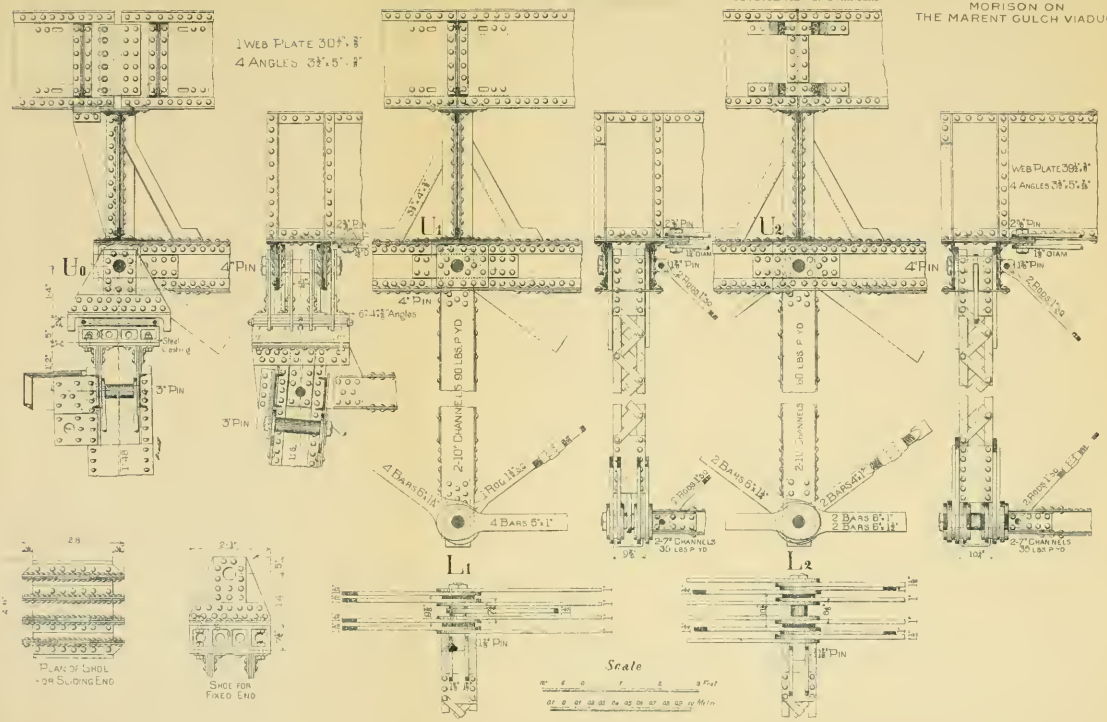
Panel Load 60,000 lbs. for each Truss, all treated as Moving Load

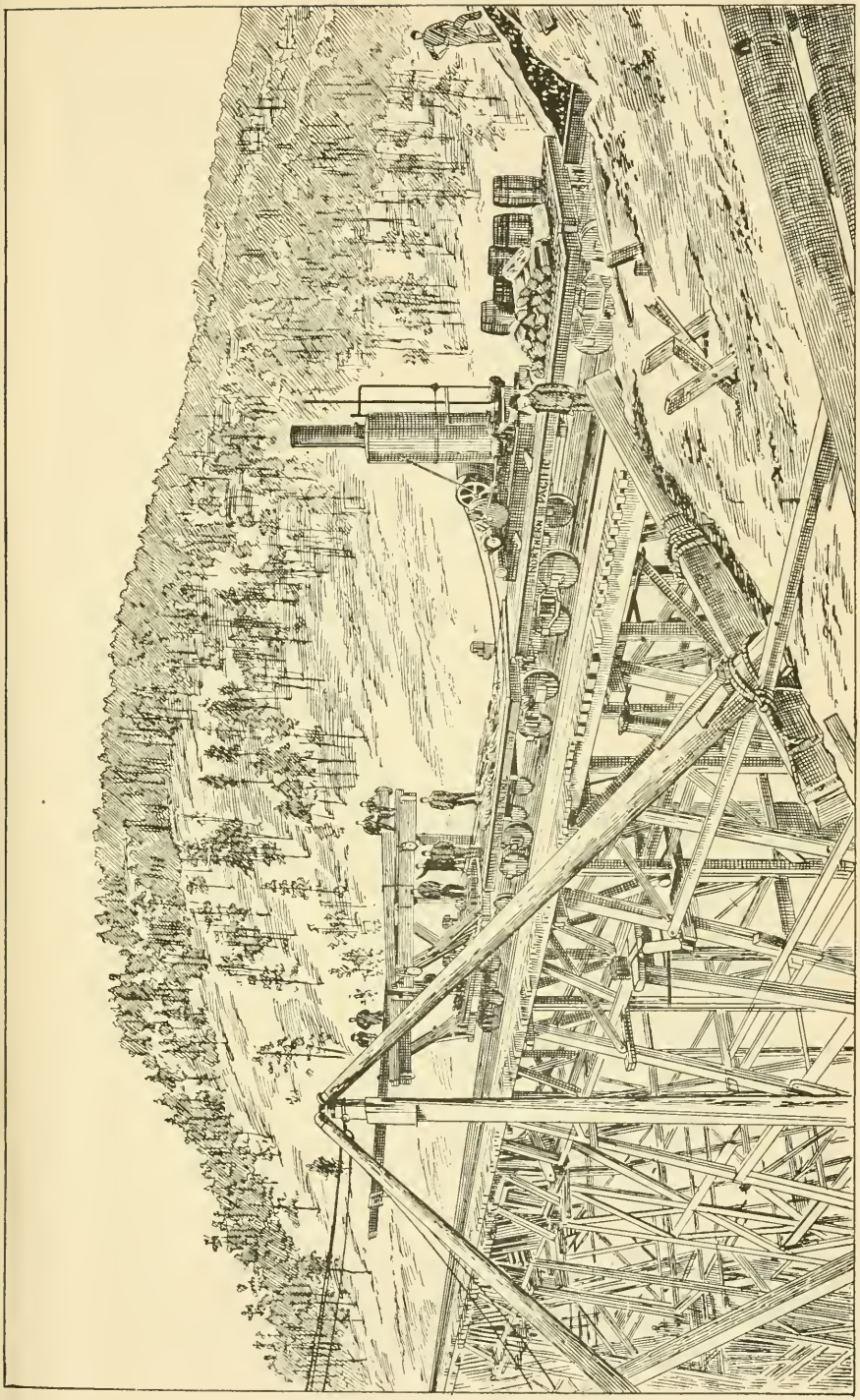
Details of Deck Span

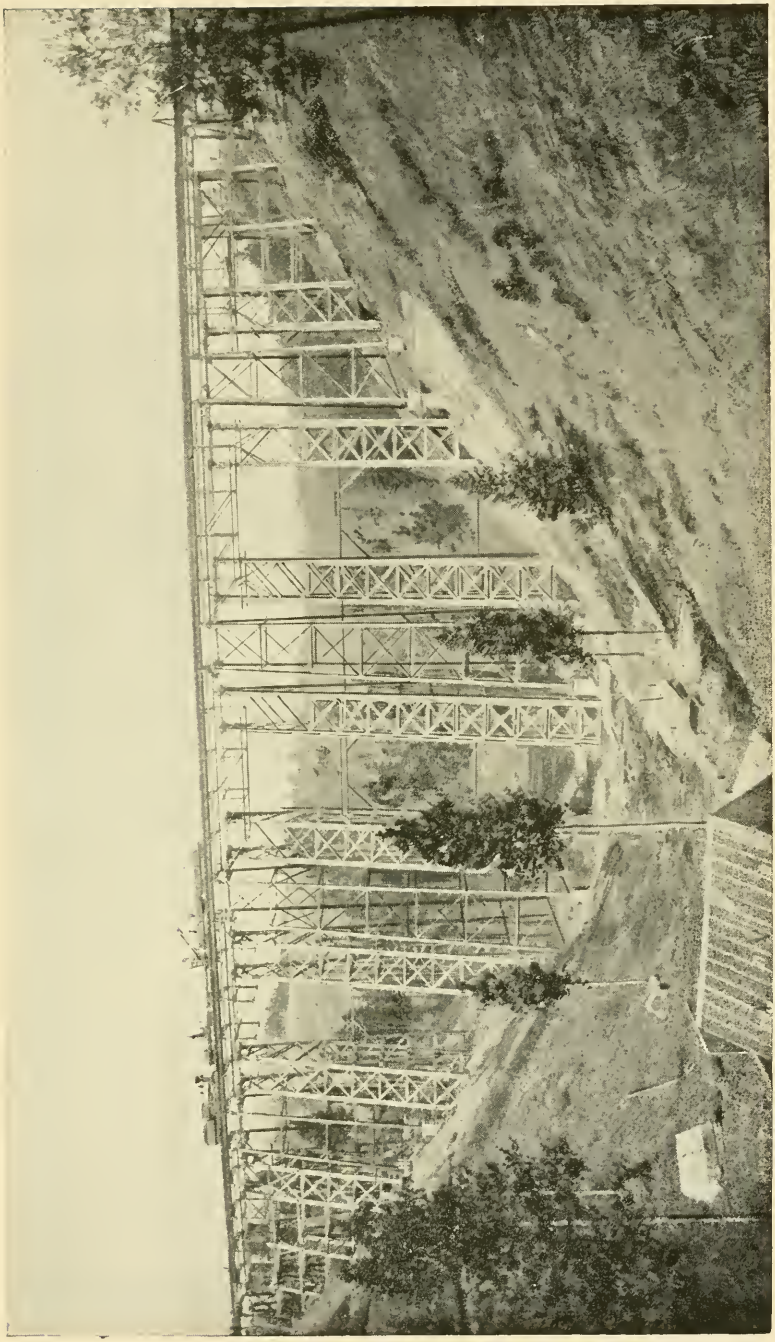
N P R R

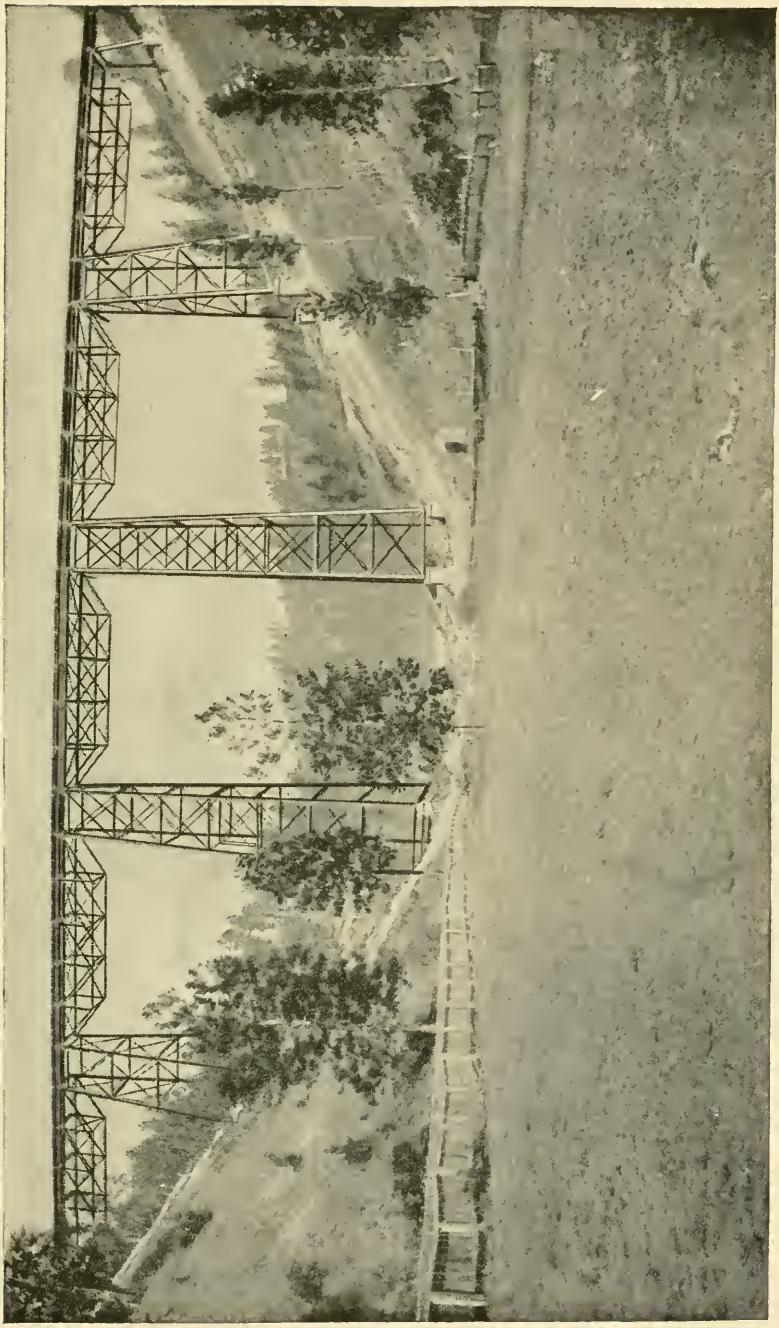
MARENT VIADUCT

PLATE LXXIII.
TRANS AM SOC. CIV ENGRS
VOL XXV NO 500
MORISON ON
THE MARENT GULCH VIADUCT




NORTHERN PACIFIC





AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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501.

(Vol. XXV.—September, 1891.)

RIGHT OF WAY FOR RAILROADS.

By JULIEN A. HALL, M. Am. Soc. C. E.

WITH DISCUSSION.

This very important department of railroad work does not always receive the attention it merits, and until recently it has been somewhat neglected. This has been largely due to the fact that property owners have generally been only too glad to promise the right of way free, in order that the railroads might be built, knowing how much the value of their property would be enhanced thereby; while the presidents of the roads have had in view the rapid, and at the same time economical, construction of their lines, and have not always taken the precaution to secure the necessary title to the property occupied. The result is, that after the lapse of years, during which the property has, by reason of the construction of the road through it, largely increased in value, the owner fails to remember that he gave the right of way and makes a determined effort to get pay for it; or, if he is forced to remember it, he remembers also that there were certain conditions in the grant with which the company has failed to comply, and, in collusion with a lawyer, enters suit against the company, generally succeeding in getting from five to ten times the value of the property taken.

The right of way for lines built during the last few years has, however, received much better attention, and in the future will probably give little or no trouble. One of the things with which we now have to contend is the considerable amount of right of way for which no settlement has ever been made, and in this, the only thing the railroad company can do is to wait for claims to be made and then compromise as best it can. It has been thought that occupancy of land by a railroad for a term of years gives it a title to the land, but a recent decision has shown that this belief is incorrect, and that a railroad can acquire land only by settlement with the owner or by condemnation. Another matter which sometimes gives a good deal of trouble is to prove the ownership, even where the title has apparently been acquired. On referring the case to the individual who is supposed to be the proper custodian of the titles to all property owned by a company, it is often found that by some carelessness he has not in his possession the deed making the conveyance, and then comes an exhaustive and sometimes rather expensive search through the court records for the information required. The best man to make this search is an engineer experienced in such work, and every road liable to have claims made against it on account of unsettled right of way should have such a man in its employ. He can be promptly detailed to look into such matters, meet the party making the claim, and compromise with him before a lawyer can interfere, and by acting promptly as indicated, the company can nearly always accomplish its purposes and save money. There are a great many engineers who refuse to have anything to do with securing right of way, claiming that such work is out of their line. It is, however, largely to the interest of the company to employ an engineer who has good business judgment, as well as engineering knowledge, to have it understood that the right of way is to be secured and put in proper shape under his direction, and to give him as his right of way agent an honest, competent, business man. This man should report to the chief engineer only, who should be empowered to make on a prescribed basis such settlements for right of way as he may deem proper and for the best interests of the company. As settlements are made, the deeds should be immediately recorded, and after the Chief Engineer has had a copy of them made in his office they should be turned over to the secretary of the company. As the deeds are received by the chief engineer he should have his right of way maps completed, copies of which, when

required by law, he will have recorded at the court houses of the counties through which the line passes. The originals should be retained for file in the chief engineer's office, and with these and his copies of deeds he will be fully equipped to give prompt and satisfactory information pertaining to all right of way matters that may arise.

As soon as the preliminary survey for a railroad is begun, a right of way agent should be put in the field, whose duty it should be to secure signatures of property owners to an agreement, providing that so soon as the railroad is constructed, a proper deed for a right of way of prescribed width shall be given. The following is a form of agreement which has been approved by first-rate railroad lawyers.

RIGHT OF WAY AGREEMENT.

"This Agreement Witnesseth: That for and in consideration of the advantages to be received by us respectively from the construction of the.....Railroad, across our lands, and in further consideration of one dollar in hand paid to each and every one of us, receipt of which is hereby acknowledged, we, the undersigned, hereby covenant and agree to and with the.....Railroad Company, that upon the location of the said..... Railroad across or upon our lands, in.....County, State of....., we will each for himself execute to the said .....Railroad Company, a sufficient deed to so much of our respective lands as shall be included between lines.....feet from and parallel to the center line of the road as it may be finally located by the company; and the agents and employees of the said company and the force engaged in surveying and constructing said railroad, are hereby permitted to enter upon said lands and locate and construct said railroad, and deeds shall be given so soon as the railroad shall be graded and track laid over each tract respectively, or as soon thereafter as shall be required."

Name of Witness. Name of Donor. [seal.]

.....

....

.....

In a section of country where the people are anxious for a road, nearly all of the signatures asked for can be readily obtained, and the agent soliciting the signatures should take in a considerable scope of

country on either side of the preliminary line, thus providing for any change of line that may afterward be found necessary.

Where there is an indisposition on the part of the land owners to sign the right of way agreement, the party making the survey should receive instructions to run several lines considerably diverging, and in doing this it will be found that securing the signatures will be a much less difficult matter. The signatures having been secured, the right of way papers should be recorded promptly in the court houses in the counties in which the signers live. This recording will, in some States, have the same effect as a deed, and prevent the signers from making over their right to the land to any one else, thus saving the company possible trouble by any change of titles or otherwise. Affidavits should be made in the agreement before it is recorded as follows:

"STATE OF....., } *ss.:*
..... County, }

"I,....., Notary Public, do certify that personally appeared before me this day,.....
.....the subscribing witnesses to the signatures of:

(Signatures,)

.....

....

to the contract to convey, hereto attached, witnessed by him and being personally known to me and by me duly sworn, duly proves the execution of said contract for the purposes therein expressed as to each one of the parties thereto.

"Witness my hand and official seal this.....day of ..... 189 .

"....., *Notary Public.*

"STATE OF ....., } *ss.:*
..... County, }

"I, ..., Notary Public, do certify that ....., the subscribing witness to the signatures contained in contract hereto attached, to convey lands to the .... Railroad, is genuine, and was made by him as witness to the signatures of parties named herein.

"Witness my hand and official seal, this ..... day of ....., 189 .

"....., *Notary Public.*"

Prompt settlements should generally be made with parties claiming remuneration for so-called damages, as while it is better to allow a few cases to appear neglected, it is generally cheaper to the company to compromise the matter than to take chances with a jury. As the location progresses, the agent should of course secure the signatures of as many property owners as possible, on each line run. As soon as most of the signatures have been secured, the work of securing deeds from those who signed the right of way agreement should be vigorously prosecuted, and every effort made to settle the cases. A good many of the land owners will regret having agreed to give the right of way, and will try to get out of it by claiming various verbal conditions and agreements entered into with the right of way agent, for which reason, if for no other, the same man who secured the signatures to the agreement, should be the one to take the deeds. He is then prepared to deal intelligently with any land owner who may attempt to plead such conditions. Where parties demur about making deeds in accordance with their agreement, the company should stand firmly on its rights and fight it out in court. One thing, however, the author would especially recommend, and this is that everything connected with the right of way be kept as long as possible out of the lawyers' hands; any one who has had experience with lawyers in such matters will appreciate this remark. The following is a form of right of way deed which has been approved as good by several prominent railroad lawyers, and it appears to cover all the points likely to arise. The plat to be attached to the deed (see Fig. 2, Plate LXXVII), should not be made wider than the deed, and should always be noted in the deed as being part thereof. (Description in this form applies to property of heirs of John Clement. See Figs. 1 and 2, Plate LXXVII.)

"THIS INDENTURE, made this ..... day of ....., in the year eighteen hundred and ....., between .... and ..... his wife, of the County of ....., of the State of ....., parties of the first part, and the ..... Railroad Company, a corporation created by and existing under the laws of the State of ....., party of the second part, WITNESSETH, That for and in consideration of the sum of ..... dollars, to the parties of the first part, in hand paid by the party of the second part, the receipt whereof is hereby acknowledged, the said parties of the first part have bargained and sold, aliened, transferred,

granted and conveyed, and do hereby bargain and sell, alien, transfer, grant and convey unto the said party of the second part and its assigns and successors forever, with general warranty, the following property, to wit: A certain piece or parcel of land, situate, lying and being in Davie County, State of North Carolina, on the line of the said North Carolina Midland Railroad, containing four and six-tenths acres, more or less, and described as follows:

"Beginning at or near Station 1349 + 50, of the center line of the North Carolina Midland Railroad, on boundary line between said heirs of John Clement and Giles Howard; thence along said named boundary line, due north, to a point 50 feet distant from, measured at right angles to said center line; thence in a southwesterly direction, parallel to said center line, a distance of 1 345 feet, more or less, to a point opposite Station 1362+25; thence in a northwesterly direction, at right angles to said center line, to a point 125 feet distant from, measured at right angles to, said center line, thence south 20 degrees 5 minutes west, parallel to said center line, a distance of 140 feet, more or less, to a point on boundary line between said heirs of John Clement and T. W. Rich; thence south 2 degrees west, along said last named boundary line and boundary line between said heirs of John Clement and Ferris Clement, Troy Clement, and John Malone, crossing said center line at or near Station 1367+52, a distance of 550 feet, more or less, to a point 50 feet distant from, measured at right angles to said center line; thence in a northeasterly direction, parallel to said center line a distance of 1 880 feet, more or less, to a point on first-named boundary line; thence due north, along said first-named boundary line, to point of beginning, it being a piece or parcel of land, through the property of said heirs of John Clement, as will be more particularly shown by reference to attached plat, which is made part of this deed. To have and to hold the said piece or parcel of land to the only proper use and behoof of the said North Carolina Midland Railroad Company, and its assigns and successors forever.

"In Witness Whereof, the said.....
and.....his wife, hereunto set their hands and affixed their seals, this day and year first above written.

"(Signed) ..... [SEAL.]
(") ..... ["]

"Witness: (Signed) .....

"State of..... } ss.:
.....County, }

"I, ..., Clerk of the.....
.....Court, do hereby certify that.....
.....and..... his wife, appeared

before me this day and acknowledged the due execution of the annexed deed of.....and the said..... .....being by me privately examined, separate and apart from her said husband, touching her voluntary execution of the same, doth state that she signed the same freely and voluntarily, without fear or compulsion of her said husband or any other person, and that she doth still voluntarily assent thereto.

" Let the same, with this certificate, be registered.

" Witness my hand and official seal, this .....day of.....189 .

.....

" Clerk.....Court.

" State of..... } ss.:
.....County, }

" I, ..., Justice of the Peace, do hereby certify that..... .....and ....., his wife, personally appeared before me this day and acknowledged the due execution of the within deed of.....; and the said..... .....being by me privately examined, separate and apart from her said husband, touching her voluntary execution of the same, doth state that she signed the same freely and voluntarily, without fear or compulsion of her husband or any other person, and that she doth still voluntarily assent thereto.

" Witness my hand and private seal, this.....day of .....189 .

" ....., J. P. [SEAL.]

" State of..... } ss.:
..... County, }

" The foregoing certificate of.....a Justice of the Peace of..... .....County, is adjudged to be in due form and according to law. Therefore let the same, with this certificate, be registered.

" This.....day of....., 189 .

....

" Clerk.....Court."

All deeds should be made up in the Chief Engineer's office, and not one should be sent out without being numbered and charged to the person to whom sent for attention. These charges should be made in a book in which sufficient space should be allotted to each deed, to allow a full summary of the case from the first move in it until its final settlement. This record should show the name of the railroad, name of the party to make deed, number of acres taken, consideration, and when

and to whom sent. A careful brief of all correspondence about each case should also be made a part of the record, and a thorough register, carefully kept as indicated, will save much time and labor. In settling for right of way, it is often the case that a much better settlement can be effected if the money is in hand to be paid as soon as the amount is agreed on; for this purpose a sufficient amount should always be in hands of the Chief Engineer, to be disbursed as he may find necessary. When this money is delivered to the Chief Engineer, he can receipt a regular voucher for the same, the wording of which should state that the amount thus advanced is to be paid for right of way expenses and accounted for by sub-vouchers to be receipted by land owners, the same to be properly approved and turned in to the Auditor, who will make the proper credit; therefore when payments are made for right of way or damages, a receipt on a proper voucher should be taken for the amount paid, as well as taking the signature to the deed. A form for a voucher, as follows, will be found to cover most of the points likely to come up afterward:

"For land in .....County, State of.....,
.....acres, as per deed of.....189 ,
from....., to the.....
.....Railroad Company, and for all damages of whatsoever nature, to contiguous lands, growing crops, by reason of borrow pits, waste banks, change of roads, streams, ditches, etc., that may have occurred or that may occur by reason of construction and operation of said.....Railroad.

"No land should ever be paid for until the title for it has been carefully examined by the company's attorney, and a certificate received from him that title is absolutely valid. This certificate, with abstract of title, should be filed with deed to which it pertains."

To cope successfully with matters pertaining to settlement of right of way, a man should have some special fitness for the work. Ordinarily the man selected is one well known throughout the section in which he will be at work, and generally such a man is either a politician or a lawyer. In rare cases such men are satisfactory, but generally they are unbusinesslike, careless, and apt to neglect points which they may consider small and unimportant, but which may, years afterward, be the very ones for which some scheming lawyer is looking. The author has always thought that a good level-headed trader and business man, one who is incorruptible (if such a man can be found), would be a far better man for right of way work than one whose principal recommendation is his acquaintance with the people. It is often the case that individuals granting a right of way desire a permit from the company allowing

them to cultivate the land on the right of way and up to the edges of the cuts and fills until such lands are required for the use of the company. There is generally no objection to granting this permission, and a form as follows is suggested as one fully meeting the requirements of the land owner and not binding the company to anything.

"..... RAILROAD COMPANY."

"I HEREBY AGREE for the ..... Railroad Company, that ....., may have the privilege of cultivating so much of the land deeded by him to the said Railroad Company, as is not now occupied by its cuts and fills and other improvements, so long as said Railroad Company may not require such lands for its purposes.

"..... Railroad Company.

"By .....

"Chief Engineer.

"Given this..... day of ..... 189 ."

The making of right of way maps for permanent record is a matter that should receive the most careful attention. The first set made up should be used as progress maps on which the date of each settlement, as made, should be entered; and as rapidly as possible, maps showing the alignment, grade, width of right of way, etc., should be finished up on tracing cloth in mile sections, and on the scale of 1 inch to 200 feet, as shown in Fig. 2. The trouble with the generality of right of way maps is that they contain too much that is useless, and too little that is useful, but it is believed that the information given in Fig. 2, is all that an engineer will ever require in such maps; and when the State law requires the filing of right of way maps in the court houses of counties through which the line passes, this will generally meet all requirements. In this connection a satisfactory and economical permanent record for the court can be made by blue-printing the tracings on linaura or prepared blue-print cloth. In this way the labor and expense of making additional maps is saved, and one may be sure he has exact duplicates without checking them over.

As the subject of right of way for railroads is one that will every year become more important, and one with which almost every engineer will at some time have something to do, the author hopes that the Members will discuss this paper fully and give their views, pointing out any defects that may exist in the methods proposed, in order that others may profit by their experience.

DISCUSSION.

ROBERT L. HARRIS, M. Am. Soc. C. E.—This is a subject of importance, which I am pleased that Mr. Hall has brought to the notice of our society, and I indorse his paper. The subject is one with which I have had much to do in various States and in Canada. My experience has been that, whenever the obtaining of the right of way has been made one of the duties of the Chief Engineer, there has been little, if any, unnecessary trouble; everything concerning it has gone along systematically and pleasantly. Whenever, on the contrary, the matter has been in charge of a man not under the direction of the Chief Engineer, there have been difficulties and trouble. All the data must be obtained from the office of the engineer, and the subject is far more distracting to his attention, and there is more likelihood of error if, in the multiplicity of details and new business attaching to the earlier months of the railway, he be interrupted at any and all times by the necessary queries and conferences about right of way matters which he, when knowing all about, would naturally settle in proper season. Moreover, the education of the engineer being to exactitude, his future reputation is at stake, right of way matters being those of permanent record. Although the original details of extra widths for deep cuts, high banks, borrow pits, spoil banks, extra tracks, buildings, etc., may be well worked up, yet exigencies will occur during construction, where an addition here and there will be of great value to the railway company. If the right of way is in the engineer's charge, this can be done promptly, pleasantly and without talk or extra stir.

Most railway engineers have, from necessity, migratory habits; yet the right of way agent usually takes his flight earlier in the fall, and then uncomfortable or perplexing questions are left for the engineer, and there is a mass of details, papers and correspondence to be searched to settle such. The engineer, being the first on the ground, it is to the interest of himself and his parties, as well as for the pleasure and the advantage of the company, that friendly relations be cultivated with people along the route. The engineer is naturally invited to the homes of land-owners, and for the time-being is one with them; consequently it is a source of chagrin to him and detrimental to the interests of the company if the first officer to follow is not of the right stripe, is a schemer, unfaithful, or even disagreeable. The matter is best settled by having the right of way obtained through the Chief Engineer by agents who appreciate his position, act under his direction, and report only to him. Mr. Hall well says the agent should be "an honest, competent, business man." Among the qualities necessary I would place first, truthfulness; there should be no misrepresentation; "fraud

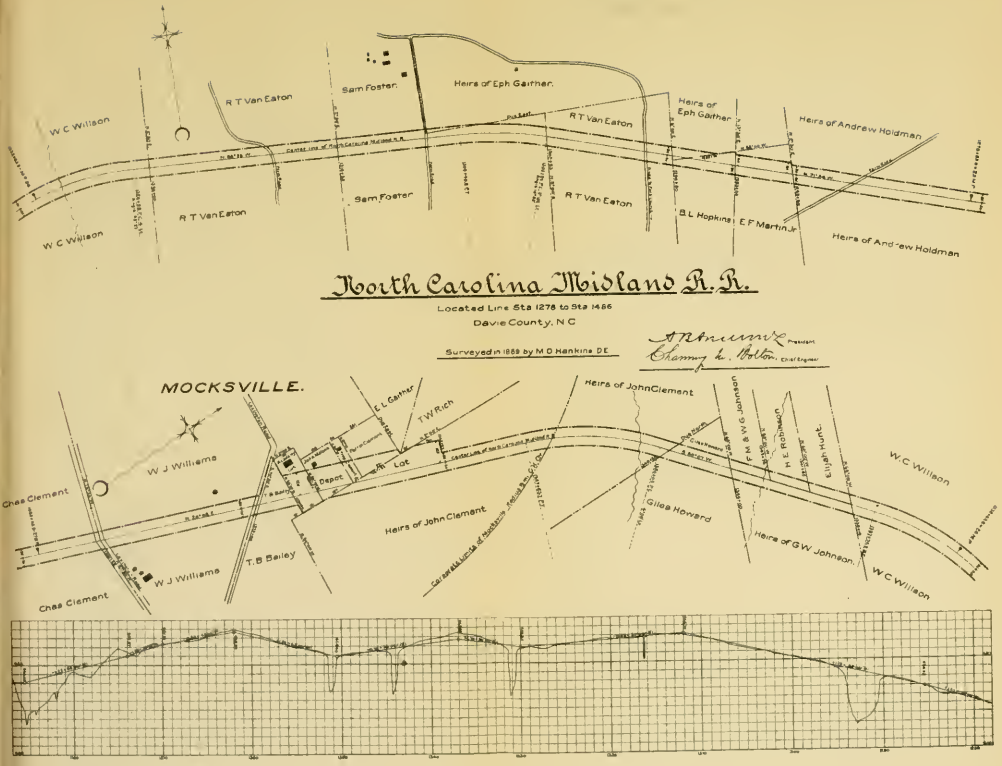
W C Willson
R T Van Eaton
Sam Foster
Heirs of Eph Gaither
Heirs of Eph Gaither
Heirs of Andrew Holdman
R T Van Eaton
Sam Foster
R T Van Eaton
B L Hopkins
E F Martin Jr
Heirs of Andrew Holdman
W C Willson
North Carolina Midland R. R.
Located Line Sta 1278 to Sta 1486
Davie County, N C
Surveyed in 1889 by M O Hankins D E
President
Chief Engineer
MOCKSVILLE.
W J Williams
Chas Clement
E L Gaither
T W Rich
Depot
R. Lot
Heirs of John Clement
F M & W G Johnston
H E Robinson
Elijah Hunt
W C Willson
Heirs of John Clement
Heirs of Giles Howard
T B Bailey
W J Williams
Chas Clement
Heirs of G W Johnson
W C Willson

FIG 1

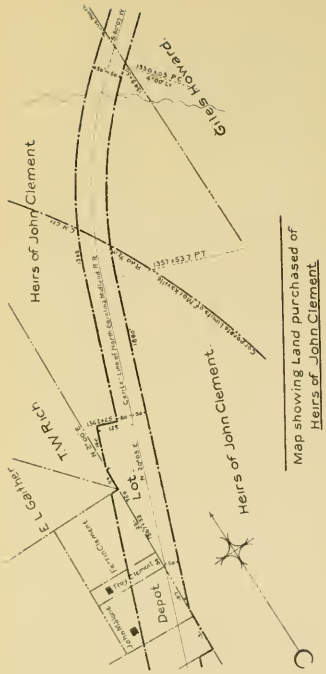
Giles Howard
Heirs of John Clement
1357+53.7 PT
Heirs of John Clement
E L Gaither
T W Rich
Lot
Depot
Map showing Land purchased of
Heirs of John Clement
by the
North Carolina Midland R. R.
in Davie Co, N C
Attached to Deed from Heirs of John Clement to the North Carolina Midland R R dated Feby 5 1891

FIG 2

vitiates a contract." Upon one road of which I was Chief Engineer, the right of way agent acted independently, and he boasted to me that he would obtain deeds for the right of way of over 50 miles of settled country for "a dollar a farm." He succeeded (he thought), but I have been informed that a half dozen years later, when the land had become valuable, some dissatisfied farmers brought suit on the ground of misrepresentation, and the right of way for the entire length of road had to be purchased at a greatly increased valuation.

When a railway is first proposed liberal grants can be readily obtained; this is the time for them to be secured. Upon a road of which I was engineer, in a well-settled, fruitful, farming region, not so far distant as Chicago, not barren of railroads, we would not accept for local station grounds a donation of less than 3 000 feet long by 250 feet wide. There was no difficulty in obtaining such when it became known that this was the standard, and at some points there was competition from the neighbors for the privilege.

In regard to the paragraph on the first page of the paper, a circumstance interesting to me occurred three or four years ago, which shows the desirability of full transit notes even in a wild country. In the company of a friend I called at St. Paul, at the offices of one of the largest railways in the Northwest. Upon being introduced to the engineer, he expressed pleasure, and said, "Your name is familiar;" he then took from the upper shelf of a large book-case a package of old note-books, handed me one, with the question: "Do you recognize that?" It was an old, worn transit-book of mine of 1858–59, when the country to the north was a wilderness and land of no money value. "That book, sir, has saved the company many thousands of dollars, the few local notes then to be had were taken with such care; and this is one of our books of reference in land and other matters."

Commenting briefly on other portions of the paper, should not the paragraph on the third page of the paper, which provides that, "So soon as the railroad is constructed a proper deed for a right of way of prescribed width shall be given," read, "so soon as the road is located?"

Naturally, in many countries, and even for right of way purposes, several lines will be run, but it is well if the Chief Engineer does not receive (these) instructions, for then the object he would attain is pretty sure to leak through others.

I am pleased that Mr. Hall makes the distinction between right of way compensation, and "so-called damages." These last should be promptly settled, whether they are from the destruction of a line of corn-stalks, the mere ravishing of a melon patch during the survey, or the greater devastation caused by the construction of the railway.

Mr. Hall says: "As the track laying progresses, the agent should, of course, receive the signatures of as many property-owners as possible

on each line run." I would suggest that this read, "as the location of the line progresses."

I agree with the statement that it is desirable to keep right of way matters as long as possible out of the lawyers' hands in the way of quarrels. This is correct; but the engineer should have the privilege of consulting freely with a good lawyer, especially at first, as the laws of the various States vary in the matter of title and conveyancing as well as in details. I may add in this connection, that in some States only right of way can be secured for a railway, and if not so used within certain periods of time, the lands revert to the previous owner.

It is generally better to describe from the center line of railroads, as has Mr. Hall, than from other boundaries, as the line in most of the States must be filed in the offices of the county clerks, and a base is thus fixed.

I do not agree with Mr. Hall in the statement that "there is generally no objection to granting permission to cultivate land on the right of way and up to the edges of cuts and fills until such lands are required." Harsh though it may seem to prevent adjoining and original owners from utilizing what is seemingly waste land; yet when a road is once built and in operation, the safety of the public is paramount. While the original engineers are on duty, and while the boundaries of the right of way are fresh, the lines of the land obtained by the company should be fenced off, and thereafter kept clean as well as clearly defined.

In conclusion, let us hope Mr. Hall's paper may tend to this matter being always placed in the hands of the engineer, and we should thank him for his effort in that direction.

P. F. Brendlinger, M. Am. Soc. C. E.—I just received my copy of this paper to-day, for which I am very sorry. It is a subject I have a great deal to do with, and it covers a very large field. I differ very radically with Mr. Hall in this right of way matter, and I propose to give my views when my notes are sent to me for correction.

Mr. Hall says, in the latter part of his paper that he hopes the members will discuss this paper fully. He has merely scratched the surface of the whole matter, and, as I said, it is a tremendous field—a field so large that I cannot commence to discuss it to-night. I saw a little illustration the other night in the Madison Square Theatre, in the play called "Alabama," which you have probably seen, of the trouble they had to get the right of way in the South. I will not attempt to discuss the subject this evening,

C. J. Bates, M. Am. Soc. C. E.—It is nearly twenty years since I have had anything to do with this subject, and I suppose that I am about as rusty as some of the railroads are. I located a great deal of the Baltimore and Ohio's Chicago line, and it was Mr. Garrett's policy always to buy his right of way in order to have a fee. He said he objected to possessing any right of way that would at any time revert. For this

reason we were occasionally delayed for some time. There was at one time a year's delay and an additional cost of $100 000 to avoid condemnation proceedings. He owned his line throughout, and, of course, his deeds were taken as Mr. Hall mentions, and everything was kept in perfect shape.

One thing I did not understand. Mr. Hall, in giving his form of deed, does not discriminate, does not show whether, when he takes the property in the shape that he gives, it would not revert in case the road abandons it, as is the case with all railroad property. Could not this be discriminated in any way ?

E. P. North, Director Am. Soc. C. E.—Does not that depend upon the laws of the State ?

Mr. Bates.—I was speaking more particularly of Ohio. The right of way obtained by condemnation will, if abandoned by the railroad company, revert to the owners, but the right of way bought, as was all the right of way by my employers, will never revert.

Mr. North.—I think, in some States, as in Connecticut, it is impossible for the railroad company to buy anything more than an easement.

Mr. Harris.—This is true, and the land will revert if not used for railway purposes.

Charles B. Brush, Director Am. Soc. C. E.—In New Jersey the condemnation proceedings extend to the right of way, not to the fee. The fee can only be obtained by agreement.

A statement is made in this paper which reads as follows: "A satisfactory and economical permanent record for the court can be made by blue printing the tracings on linaura or prepared blue-print cloth." These preparations are not satisfactory for permanent records. I think a mistake is made in registry offices when they accept such prints for records. Records of permanent value like this should be made by hand rather than by blue prints.

The right of way agreement, given in the paper, is a better thing for the railroad company than it is for the property owner. It gives the railroad company permission to locate its road where it pleases. While this may be possible in some parts of the country, it is not possible in this locality. When an owner gives to a railroad company "a sufficient deed to so much of his lands as shall be included between lines * * * feet from and parallel to the center line of the road as it may be finally located by the company," he simply gives the company a roving commission to take possession of his farm. In many sections it would be impracticable to obtain such a deed.

I think it is an improper description to locate a piece of land from the center line of the tracks, because these are shifted from time to time. When a railroad goes through a farm, for the safety of the railroad people and for the safety of the owner, the point where the road crosses the boundaries should be located from given land marks. I think the ten-

dency of the paper is to look at this matter more from the standpoint of the railroad than from the standpoint of the property owners. I believe the only way that a satisfactory location can be made, where property is taken for public purposes, similar to this, is to make the location and arrangement with the owners in such a way as will be mutually satisfactory to both. I do not believe that can be done unless the agreement is definite. Generally it will be found that, unless the farms are large, it is better to buy a farm outright, or buy a portion of it rather than it is to go into detailed descriptions for buying strips. Where railroads are necessary for the development of a given locality the right of way may be obtained easier than in this section. Any one attempting to proceed on the lines laid down in this paper in this portion of the country will almost certainly fail.

The question of the right of way being referred to the Chief Engineer is very important. I think it should be so done. On the other hand I think rather too much is made of the lawyer side of this question. I am very glad to have a competent lawyer work in connection with me. If the lawyer is not properly fitted for that kind of work he would not be of the same advantage that he would be otherwise; but I think it is a serious thing for engineers to go too far outside of their lines, where the question of legal and vested rights are concerned. It is right and proper that the lawyer should understand that his part of the work should be confined within certain limits; the same is true of the engineer.

J. Foster Crowell, M. Am. Soc. C. E.—I am much more inclined to take the view that Mr. Brush has taken than those of the author of the paper and the first gentleman (Mr. Harris) who discussed it takes. The difficult parts of questions of titles seem to me to have very little to do with engineering requirements, and sooner or later legal knowledge is necessary; if not secured at the outset it will land the company in misunderstanding later.

The author of the paper has not in his mind, I think, a real estate lawyer, but rather the employment of a lawyer as a right of way agent. It is absolutely necessary, I should say, that the Chief Engineer should have the direction as to what property is needed. He should also have matters connected with the preliminary negotiations sufficiently under his control to secure the interests of the company as regards its relation to the adjoining owners. Often a right of way agent has too much latitude given him, and he embarrasses the company with his heedlessness of requirements. I have in mind a very zealous man in this business, who was always looking after his reputation as a buyer, and I have known him to sacrifice the interests of the company in the future for the sake of keeping down the schedule of first cost of right of way. In one case, in a farming district, in order to accomplish this, he had given so many cattle-pass and bridge crossing privileges to the owners that while he had a small right of way account he had added an average of several

thousand dollars a mile which the construction department had to supply. In most cases the conditions were not required by the owner, but were offered by the right of way agent in lieu of cash. The resulting loss to the company as appeared afterwards in the construction account was very large in the aggregate. It goes without saying that these matters should be under the control of the Chief Engineer. When that is done, when the basis of these negotiations is settled in each case, the business should be turned over to the legal representative of the company as a matter of course and necessity.

Mr. Hall might find in some new country a scope for the intervention of the engineer which he thinks necessary, but in a settled country the engineer, if attending to the right of way matters, would have very little time for the construction of the road. He has in mind an engineer with a small staff, which is a very different state of affairs from that of one of our great railway companies passing through a settled country, where the real estate department takes an immense amount of time. Any one knowing of the real estate department of the Pennsylvania Railway, for instance, will see that this is a matter that the engineer engaged in construction or location cannot possibly attend to in a satisfactory manner to his company.

A Member.—Circumstances in different States seem to differ widely. I have in my hand a slip of paper, giving a very recent decision by Judge Daniels, of the General Term of the Supreme Court. Several years ago the railroad company with which I have been connected, purchased by agreement with the owner a piece of property in this city. That property, with other property surrounding it, has since been designated by the City Commissioners as a public park, and for various reasons the construction on the part of the railway has been delayed, and now the company are anxious to construct on the part that has been condemned by the Park Commissioners, and find themselves debarred from it.

Mr. North.—I think all agree that an engineer who starts out to acquire a right of way unaided by legal advice would be as apt to come to grief as the lawyer would who started out to build a railroad without consulting an engineer. As far as I know, where a road employs a great legal light and sends him out to contract for right of way—not reporting to the Chief Engineer—the road is liable to come very much to grief in its right of way matters. It seems to me it has worked better in several cases that the lawyer should be employed by the Chief Engineer and report to him or to the Assistant Engineer in charge of the work. By this means he is put into immediate communication with the man who knows the line the best; he is prevented from making any foolish contracts. In addition, you can generally rely on the engineer being honest; you cannot rely entirely on the lawyer, because he is apt to have a side issue with the people of the country, and does not devote himself so fully to the interests of the company as the engineer does, who has no friends in that country to reward or enemies to punish.

I want to emphasize what Mr. Brush says about the contract for right of way. Mr. Hall says you must have an honest man. I don't understand you can get a man who is honest on one side and dishonest on the other.

Where land is worth a great deal it is best both for the railroad company and the land holder that the points should be rigorously described, but in wild land I do not think it is necessary or desirable. It does no damage to a land holder, with five or ten dollar an acre land, to describe the right of way by its distance from the center line. If such land ultimately gets a value of $500 or $600 an acre, it is probably the railroad which has given that value to it. The company, while it does not swindle the land holder, should prevent as far as possible any swindling suits against itself. If an effort is made to locate from section corners, you are liable to find that a mistake has been made by the engineer and surveyor, and you have a great deal of trouble; but by referring the boundaries of the right of way to the center line, there is no chance for any trouble that I have ever known. The average corner in lands of low value is often undetermined and its true position is not established until the value of the surrounding land justifies a suit.

Charles E. Emery, M. Am. Soc. C. E.—My experience in this line dates back some thirty-three years, to a time when I was a student in an engineering office in Central New York, where railroads were being built under the direction of Marvin Porter, Esq., and when E. S. Philbrick, M. Am. Soc. C. E., was his assistant engineer. Afterwards I was a student in a real estate law office, and I must say that the methods which were then employed in Central New York were nearer those stated by Mr. Brush than those referred to by Mr. Hall. It is true, as has been said, that the methods of procedure must be different in different parts of the country. In sections comparatively undeveloped it may be practicable for an engineer to take blank contracts with him, fill them up and have them signed as the location progresses; but in the older States, where the farms are comparatively small and every foot of land is known to the owner and in some sense utilized, it is absolutely necessary to conduct the whole matter from the start carefully, in accordance with the business methods of the particular locality. My recollection is that at the time referred to the matter was in charge of the attorney of the company and of the Chief Engineer, under the general direction of the president; and that after the names of the towns to be reached had been fixed, the Chief Engineer usually laid out the line where he desired it, and a local attorney was employed in each county to deal directly with the land owners, secure the execution of the necessary conveyances, attend to the recording of the same and the filing of necessary maps, and in general to smooth the way in all directions, in conference with the engineer and other executive officers. The employment of local talent had the effect to make the road popular,

and a person who knew the individuals could deal much better with their personal peculiarities. Slight changes desired here and there were generally adjusted by conference. The counsel of the company arranged that all conveyances were in satisfactory form before approving the payments, and on the whole the business was conducted in customary business ways, differing, as will be seen, very little from that in which any private individual would arrange for the purchase and secure the transfer of a single piece of property. Obstinate men were found, of course, but as the business was conducted so as not to cause public disapproval, the more difficult cases were generally solved with comparatively little difficulty.

George R. Hardy, M. Am. Soc. C. E.—I want to call attention to one feature of the paper under consideration. The scale mentioned as desirable for plans to be filed is 200 feet to the inch, and it seems to me that in most of the east, at least east of Chicago, if the land is of any value at all it is relatively too small a scale. The desirability of describing the property in the manner which is suggested, at distances from a center line, is manifest, but a very short description, avoiding going into inside meets and points, is necessary in order to locate city lands. That is rendered necessary in places where land is bought and sold by the foot; with some lawyers I have found that there was no other way of satisfying a railroad form. The best results were found by describing the land bought and required by the railroad company, but as soon as one goes outside of villages it is sufficient to describe the land by the distances from the center line, and in no case have I found it satisfactory to describe the distance from a center line which has not been located; the result in that case is too indefinite to have a permanent value. If the railroad company has located its line and established it, the line should be so located by reference points that it can afterwards be revived. I have had a great deal of business in the course of reviving an old location, and the difficulties have been in obtaining property that is described "as now staked out." The location should be well defined and at tangent points some permanent record may be placed, or if there is any masonry in the construction at that place, getting offsets from the center line, then the widths at each side can be revived at any time. But it is necessary in all such cases that the right of way maps should be of such a capacity as to admit of writing the exact widths without crowding the plan at all. Also to be able to locate the masonry features of the road by distances and offsets from the center line, such as the bridges, cattle guards and culverts; this is necessary, and once having these located as above, a great deal of the difficulty of reviving the old location is eliminated.

Mr. Brush.—If the center line is well defined and made a matter of permanent record, I do not think it necessary to go into the question of meets and bounds. I referred rather to cases where the location of the center line was indefinite.

Mr. CROWELL.—It would be impossible to lay down hard and fast rules that would apply universally. In new countries, where securing the right of way is a necessary preliminary to starting the enterprise, it is often necessary to obtain a release first of all, the release having a binding effect on the owner; the deeds are afterwards prepared in conformity with it, but with a description resulting from the final surveys. It is quite sufficient for such purposes to define in the release a "strip of land" which shall "conform to the line as it may be located." But in thickly settled portions of the country, where land is valuable and all taken up, there is hardly ever a necessity for this haste, and negotiations can be begun on a basis determined by actual location and determination of grade. I think we are rather talking around the question in going so much into details, because they would vary in nearly every case; what would be proper in one case would not be necessary in another. I think Mr. North, in his remarks as to the advisability of a little looseness, does not really conserve the interests of the railroad company; it might possibly find itself in trouble on account of the looseness of the description; where possible, it is very much better to recite in the deed the connection between established points and the center line, but the description should be built upon the center line invariably.

Mr. BATES.—It may be interesting, as a matter of precedent, to say that, in the case I speak of, all employees of the company were absolutely forbidden to make any promises to the land owners; and in many cases, where an end of a farm was cut off, that end was purchased from the farmer and afterwards sold back to him at a reduced rate, the difference in cost being in favor of the railroad.

Mr. HARRIS.—In the more unsettled parts of the country the center line is fixed, it is recorded in the county clerk's office, and it is readily retraced at any time, so that, except in very valuable lands, it is generally most convenient to describe by using the center line.

Mr. WILLIAM P. SHINN, Past President Am. Soc. C. E.—This paper of Mr. Hall's is one of very considerable and rapidly increasing importance. The great increase in value of property which has been brought about by the construction of railroads has become a source of considerable expense to the railroad companies, because when they were constructed it was not thought necessary to properly secure the right of way. There are some points that have occurred to me as quite important that have not been mentioned by Mr. Hall. Mr. Hall refers to the fact that legal decisions have been made to the effect that a railroad company cannot acquire right to land by occupation. It was thought for many years that the law of limitation, which operates against private claimants of land, would also operate against the previous owners of land occupied by railroad companies. I think the first decision upon that point was made by the Supreme Court of Pennsylvania, in which

they decided that inasmuch as private property taken by railroads is "private property taken for public use," and as the Constitution provided that "private property should not be taken for public use without compensation," that the property could not be considered as taken until the compensation was made, and so, in fact, although the property had been occupied by the railroad company for a period longer than would have sufficed to have given a private party in possession the ownership of the land, the court decided that there was no ownership by possession.

Mr. Hall refers to the importance of making right of way maps and having them recorded, and that the originals should be returned and filed in the Chief Engineer's office. We all know what is meant by "filing" a plan. It means to put it in some drawer or closet, and when it is wanted the chances are against its being found. Instead of being satisfied with merely "filing" the plans, the proper course to take, in my opinion, is to make these plans on sheets which shall be bound in books; those books are not easily mislaid. In the last four railroads that I have had charge of (in neither of which had any right of way maps been preserved in that way) I had maps of the right of way made in this manner. Each county was made the contents of a book; the first page of a book—the pages being about 20 x 40 inches in size—contained a map of the alignment and property lines in a general way, within the county, drawn to as large a scale as the size of the sheet will permit. Then followed on the next page the alignment and right of way lines in the first township passed through in that county; then on successive pages the right of way maps prepared on a large scale, giving one or more on the page, according to the length of line in land owned by each person. The maps of the county being completed, were bound together, paged and lettered on the outside so as to be easily referred to. In addition to those maps I had also a record book in which the descriptive parts of the deed, or condemnation paper, or whatever title the company had, were entered plainly, being copied verbatim, with a reference to the book and page of the right of way map. This book, being indexed, furnished practically an index for both books, and in that case any particular right of way and the character of the title held by the company could be ascertained in a very short time. If that were done immediately after a railroad was built, or while it was in process of construction, it would be seen at once whether the title was positive or defective. In making up these maps and records on the roads to which I refer, we discovered in quite a number of cases where it was supposed the company had the right of way, that there was no complete title.

There is another subject which is referred to by Mr. Hall in his paper, that is, the importance of having a good business man to secure the right of way. While it is proper for the engineer to locate it and mark it out upon the ground, as a general thing the engineer is not the best party to make the bargain for securing the title; but one difficulty

with the average right of way agent is that he expects to be held responsible for the sum total in dollars and cents only of the cost of securing the right of way, and if he comes to an actual case in which it will cost $1 000 for a road across the track, or a cattle pass for the payment of $50 or $100 in cash for the right of way, as the expenses of road or cattle pass go into the construction account, they will be allowed to outweigh in his mind the advantage of securing the right of way for $50 or $100 in cash, so that it is true that the right of way agent should not be allowed to make any bargain without the approval of some officer of the company. Mr. Hall suggests paying the money into the hands of the Chief Engineer for the purpose of making some settlements. In that I do not agree with him. I think the Chief Engineer has enough to do without handling money or making settlements, but he can make a contract in some cases and he should make his proper voucher and certify it to the proper disbursing officer of the company.

There is one thing more: not only is it important to secure the right of way by deed or other form of title, but it is especially important to secure the exact location of that right of way. The importance of that matter has been brought very strongly to my mind within the past two weeks, in which I have been called upon to give some information and probably to testify in court, as to how the center line of the railroad that I was on and engaged in the construction of forty-one years ago was located with reference to the tracks. The title to the right of way in that case is all right, but the question is, where is the right of way? A man has built a stone wall in such a position that the railroad company claims it encroaches upon the right of way, and they wish him to take back his stone wall, but the man replies: "My stone wall is on my own land; I have such and such boundaries and my wall is on my land, and you take your old railroad and put it on your own land." In that case there is no possibility of absolutely establishing the exact location of that original center line, and that causes me to remark that as soon as the preliminary location of a road is made, every point of change in the alignment should be located by good reference marks and on long tangents (such as we have on many western roads); points in the tangents, probably every half mile, ought to be so located, when the road is built; before these reference marks perish, good substantial monuments should be put in the road-bed, marking that center line. We are in the habit of getting up monuments marking the lines of much less important property than that, and yet I presume that not more than one road in fifty of the railroads of the United States has any permanent marking of its center line.

Robert Moore, M. Am. Soc. C. E.—I would like to add my testimony in confirmation of what Mr. Shinn has said as to the great value of preserving papers. In my experience there is no way of doing this except to bind them. As a rule, the unbound sheet disappears in a very

few years, and if, as in the ordinary custom, the engineer himself is dismissed as soon as the road is opened, it is very sure that his loose sheets will be scattered in a very short time. Every one should make it a rule to see that they are permanently bound so that they can be easily preserved.

GEORGE B. NICHOLSON, M. Am. Soc. C. E.—As to the cost of getting right of way after the track is laid, I can cite an instance in my observation. I had in my possession a deed very carelessly made out by a right of way agent, who obtained the right of way some years since through a farm by a deed containing a condition which required the railroad company to keep all drainage to one side of the track, although naturally it should cross it. The land owner did not ask for the condition to be fulfilled until fifteen years had elapsed. A survey revealed the fact that it would cost about $3 000, or as much as the farm was worth, to keep the drainage from crossing the track. The land owner was then offered a cash consideration of $300 to take care of the rain-fall himself, which offer was accepted with alacrity, and for five years the water has continued to flow through the box culverts under the road-bed.

I have in my possession the deeds of one of the first railroads built in the south, which deeds date back to the year 1835. I suppose the projectors and builders of this road had had no experience in railroading, and their ideas of how they would operate it were somewhat vague. They must have thought that it would be conducted after the manner of a canal; that is, that the company would furnish the railroad and the patrons the cars, for the consideration in one deed for the right of way was that the railroad company was to provide a "burden" car whenever required by the grantor or his successors and assigns. One company succeeded another as in the usual fortunes of railroads, but the "burden" car had never been furnished, and after the lapse of nearly fifty years the son of the grantor sued for the performance of contract and damage for past neglect. The decree, which was sustained by the Supreme Court of the State, awarded the plaintiff $15 000 for past damages, and enjoined that the company should provide the car for all future time whenever demanded. The $15 000 was not paid, as the road was sold at a receiver's sale shortly after the final decision, and the debt was thus wiped out; but the irksome stipulation of furnishing a car still continues in force.

With the object of avoiding the obnoxious condition, a survey was made for a new line around the farm, which was only half a mile wide along the track, but as it extended one mile on each side and as the original line had been well located on the best ground which could be found in a hilly country, the cost of building a new road was so great that the subject was dropped and the railroad company now furnishes the "burden" car.

I disagree with Mr. Shinn concerning the control the Chief Engineer

should have in purchasing the right of way. Of course he is a very busy man and cannot personally look after the details, but he should have the supervision of the right of way agents. If this has not been the case in the original construction, the engineer or his successor in charge of the finished road will have years of labor in eliminating the mistakes of right of way agents. I speak from severe experience.

In making out deeds I think the legal department and the engineering department should work together. It has been my fixed rule, no matter how simple the deed may be, to insist upon writing the description myself, and that the lawyer shall examine the deed, to certify to its correctness as a legal document. I have frequently to deal with trouble in right of way on account of the looseness with which descriptions in deeds have been drawn.

Mr. Shinn.—I wish to correct the impression of Mr. Nicholson that I mean to convey the idea that the right of way agents should not be subordinate to the Chief Engineer. I merely said the Chief Engineer should not have to attend to the business himself. I do not think the Chief Engineer should have the disbursement of the funds, but he should approve all contracts for right of way before they become binding.

Fred. Brooks, M. Am. Soc. C. E.—In Massachusetts the Board of Railroad Commissioners has established rules in detail for the systematic mapping of railroad locations for record, in pursuance of Section 93 of Chapter 112 of the Public Statutes, which reads:

"The Board shall prescribe such rules as it may from time to time deem necessary in regard to the form in which all records of locations of railroads shall be made, the particulars to be contained therein and the manner in which such records shall be uniformly kept for preservation and convenient reference in the offices of the clerks of the several counties. No such record shall be filed until the Clerk of the Board certifies thereon that the same is prepared in conformity with the rules prescribed by the Board."

Mr. Brush.—The company with which I am connected does not accept any deed until the engineer and the counsel of the company have actually indorsed thereon the fact that they have approved the description and the form of the deed. This action has been taken during several years past and the result has been entirely satisfactory to the company.

John F. Wallace, M. Am. Soc. C. E.—Engineers engaged upon railroads will appreciate Mr. Hall's efforts to attract proper attention to this department of railroad work. During the rush of construction the first thing considered is, usually, how immediate possession of the right of way can be obtained, and it is seldom that care and attention is given to the matter of procuring the execution of the proper deeds and papers. This is a most prolific cause of trouble.

During the location and construction of a line of railroad built sev-

eral years since, of which the writer was Chief Engineer, the right of way agent reported to the Chief Engineer and followed the location party closely. Each night the agent was furnished with sketches showing the location of the line, and instructed as to what and how much variation from the located line should be mentioned in the right of way contract; also as to where under or over crossings could be provided, and any other special conditions which would enable him to act intelligently in making contracts. The right of way agent was instructed to close contracts with each landowner at first interview, if possible, if not to inform the landowner that condemnation proceedings would be immediately commenced. All right of way cases that could not be promptly settled were at once placed in the courts for adjustment. Upon completion of the road deeds were required and all titles perfected; when differences arose that could not be arranged suits were instituted to perfect title.

Upon the completion of construction, the right of way map should show all lands belonging to the railroad company, the location of monuments and the character of the title, also the names of previous owners. A record book should be kept in the office of the Chief Engineer containing an abstract of every deed, a description of the land, references to the right of way map, and all conditions mentioned in the deed or agreement.

During 1889 and 1890 the writer was connected with the construction of a railroad terminal into the City of Chicago. Our line was located approximately on existing maps and the securing of the right of way placed in the hands of a real estate agent, who acted as trustee for the different roads interested in the terminal. Before the public were aware of the construction of the road, over two-thirds of the right of way had been secured by deed. In all cases entire lots were purchased and parts not needed afterwards sold. After securing a city ordinance in which the route to be taken by the railroad was designated, all unsettled cases were placed in the courts and title secured by condemnation. The values placed by the courts upon the land taken were reasonable and fair, and generally within the amounts previously offered by our right of way agent.

The right of way agent should, in all cases, report to and be under the direction of the Chief Engineer. Trouble is often caused by the right of way agents agreeing to conditions and assuming obligations, which the engineer in charge of construction may find unwise or impossible to carry out. The result of this is litigation and the making of enemies among the future patrons of the road.

The writer, some ten years ago, had charge of the construction of a terminal line into a western city. The line was located for a distance of three miles along a river front between various industries and the river. In one case a large elevator had constructed a loading dock out

into the river and connected it with the main elevator by various carriers, in order to load and unload vessels at the dock. Our line was located between the dock and the elevator, and not being able to reach any agreement with the elevator company, condemnation proceedings were instituted. The railroad company filed plans showing how the railroad could be constructed and how the elevator carriers and connections could be re-arranged to admit of the joint use of the property by both parties, and requested the court to assess damages, if any, upon this basis. The elevator company endeavored to convince the court that, while the railroad had the right to condemn the fee to the land needed, the condemnation must be absolute and not partial. The object of this stand upon the part of the elevator company being to obtain a verdict for such excessive damages against the railroad company as to prevent the construction of the road, the elevator company being owned and controlled by a rival railroad company. The court held that the railroad company asking for the right of way could file plans and prove its intention to construct the road in accordance therewith by the testimony of its engineers; and that the court, to determine the amount of damage due the elevator company, should consider the effect upon the elevator company, of the construction of the railroad in accordance with said plans. This decision was afterwards affirmed by the Supreme Court of the State of Illinois. The same ruling has since governed where railroad companies have condemned their right of way through farm lands, and filed plans to provide for under and over farm crossings, in order to show a minimum damage to the property owner.

Annexed is a simple form of right of way agreement, which the writer has frequently used and found very satisfactory. Separate agreements are preferred with each landowner, as it is seldom that signatures to a general agreement can be obtained.

RIGHT OF WAY.

"In consideration of the benefits to be derived from the location and building of the....., and one dollar to me in hand paid by said railroad company, the receipt whereof is hereby acknowledged, I do hereby grant and convey unto the said..... ..... the right of way for said railroad..... wide over or across the.....
.....
.....
.....
.....

"And I promise and agree to make all proper and necessary deeds to convey in fee simple to said company, said right of way, as soon as said railroad is located on or across said above described lands or lots.
.....
.....
.....

"Witness my signature hereto, this.....day of ....., A. D. ....."
.....

H. V. Hinckley, M. Am. Soc. C. E., writes : A certain amount of system as to deeds and maps is both essential and economical. The following suggestion will show the system that I have used on the Santa Fé road for the past ten years as the result of previous experience. Deeds for all grounds except for a uniform width across a section or quarter section are made out in my office and sent usually to the right of way agent. Attached to each deed is a memorandum slip or tracer as follows :

Acres, 10.6..... At Garden City..... H. V. H.'s No. 1376
Price, $100..... Total, 1 060..... For Stock Yards..... By J. E. L..... Check E. N.....
Title O. K., Abstract 137..... Owner. P. Jones..... To C. B. K., 3/19..... 1887....
Sent O. K. to..... 188.... Abandoned for.....188....
....

TO THE PARTY NEGOTIATING :

When deed is secured, or for any reason abandoned, fill the above blanks, note hereon any changes that have been made, sign on lower line, and send this memorandum promptly to—

A. A. Robinson,
2d Vice President and General Manager,
Topeka, Kansas.

For H. V. Hinckley,
Office Engineer.

This tracer is copied with the deed in an impression book, and when the slip comes back to us the notation is promptly entered on the impression and may read as follows : "Sent O. K. to Treasurer 3-20-87," or "Abandoned for condemnation 3/20/87, all papers to law department." Each deed also has my number on its face, so that any error made in my office can be traced to the man who wrote the description and the man who checked it, and by these slips any lost deed can be traced to the loser.

The deed is sent by the right of way agent with the voucher to the treasurer. The treasurer gives the deed his consecutive number, and it then comes to the Chief Engineer's office for certification (on its back) and entries as follows :

In the right of way book A, B, C, or whichever book has the deeds of the line to which this deed pertains.

In the name index book.

In the numerical index book.

In the right of way map or station plot, correcting the same to date to correspond with the deed.

All deeds must be sent by the treasurer to the Chief Engineer when they come in. All deeds reaching the Chief Engineer must be promptly entered as above and returned to the Treasurer.

As to blue printing I cannot see how a road can afford to be without an apparatus for this purpose. We have four frames, 3 feet by 9 feet, capable of printing 5 000 yards per month, and we have run as high as 3 600. Also one frame, 4 feet by 5 feet, for old odd sized drawings. No drawings are now allowed to be made over 36 inches wide. A boy runs the four frames, and has a helper during times of construction.

DEED RECORD BOOKS A. B. C., &c.

T 21 S. R. 13 E.

Tract.	Description.	Grantor and Dates.	Conv. and Consid.	Number.
N. E. ¼ of 34	50 feet each side of center line. Also, 50 feet Extra each side from station. 260 × 40 to station 280 × 16. Printed alike on both sides.	John Smith *et al.* W. D. ex. 3/20/87. Rec. 3/21/87.	Warranty, 1 200.00	11348

Numerical Index.

Trans. No.	Date Executed	Conv.	Grantors.	Brief.	Area.	Consideration.		Sent To.				Book.	Page.
								Tr.	M.	D.	Y.		
11347	3-20-87	W. D.	John Smith *et al.*	100' across N. E. of 3-50-13 and extra.	12.00	1200	00		3	30	87	K.	113
11348			Printed alike on both sides.										
11349													

C. D. Purdon, M. Am. Soc. C. E., writes: I would suggest that the number given the deed be written on the right of way map, so that the deed could be quickly looked up, and that the right of way be plotted on a line on the bottom of the profile, when the profile is separate from the map.

Mr. Hall says nothing about "Cattle guard releases," which are agreements whereby the land-owner, for a consideration, releases the company from the obligation of building cattle guards on the land. These are pretty much the same as a deed, and should be recorded in the same way.

Julien A. Hall, M. Am. Soc. C. E.—I am much pleased to see that Mr. Harris has endorsed my paper, and I am sure he fully appreciates the many difficulties and troubles to which a man in charge of right of way is subjected. I can fully understand why his old note book, containing probably the only reliable information extant, was considered so valuable by his engineer friend in St. Paul, and it is to be regretted that many engineers do not consider it necessary to be thus exact. I suppose the reasons for this are, that some have not been properly trained and others rely very much on their memories. This latter, for present purposes, is very well; but when a man receives a company's money for his services it should be his earnest and constant endeavor to render a proper return for such compensation, and good notes cannot be too highly valued, and, as Mr. Harris' friend says, often save the company much money.

It would be well, I think, as suggested by Mr. Bates, for a clause to be inserted in the form of deed I proposed, to the effect that the land occupied by the railroad be not allowed to revert to the original owner in case of abandonment of the line, and in such case this clause would certainly save the company much money. And I think this clause should be inserted in deeds used in all States regardless of the laws of the States in which the deeds are used, as no one can tell how a lawyer may show this matter to a jury, and where there is a possibility of dealing with gentlemen of the legal profession, it is well to put a safeguard around every outpost.

I note Mr. Brush criticises my statement that a satisfactory and permanent record can be made with blue print maps. I am loth to disagree with this gentleman, and will merely say that I have had experience with blue prints only during the last ten years, and during that time I have never known one to fade. Of course, tracings made with India ink are known to be a permanent record, but my experience shows blue prints to be fully as lasting. I note also that Mr. Brush thinks that obtaining the "roving commission" given in my "Right of Way Agreement," of "a sufficient deed to so much of his lands as may be included between lines ——— from, etc.," would be impracticable. In this connection I would merely say that Mr. Brush's objection will,

without doubt, apply in some sections, but it is nevertheless a very good thing, and where it cannot be obtained the next best arrangement must be made. But this roving commission can be obtained in many sections and has been found very useful. Mr. Brush is, no doubt, also correct when he says, "Any one attempting to proceed on the lines laid down in this paper, in this portion of the country (meaning New York State, I suppose), will almost certainly fail." To this I will say, that during the last five years I have had considerable to do with getting the right of way on nearly 250 miles of railroad line in the South. We have proceeded as indicated in this paper, and have had little or no trouble in closing most of the cases satisfactorily, except where we were handicapped by the law's delays.

I think if Mr. North will read my paper more carefully he will see that I did not make it conditional that the right of way agent recommended by me should be honest only for the company, and I cannot understand why he should take this view of my remarks. I, too, think that no false promises should be made to the land owners, and that they should in all cases be fairly and honestly dealt with. But a railroad company, as is well known, is always looked upon as fair game, and the land owners will, as I said in my paper, on account of some imaginary grievance, sometimes go in "cahoot" with a lawyer and succeed in getting from five to ten times the value of the property taken, said land having been made valuable by the building of the railroad over it. A railroad generally has its hands pretty full to keep from being made bankrupt by people who abuse its property and take every advantage of it. It appears to be a fact that there are a few men managing railroads and all the world is trying to get the better of them, and a good many, by hook or crook, succeed in so doing. It is not my policy to be honest for the railroad company alone, but I look first for its interests, in each and every point, in every case that comes up; and it is the duty of others to do the same for themselves.

I agree with Mr. Charles E. Emery, that the employment of local attorneys to assist in procuring right of way will, to some extent, make the road popular, provided a suitable man can be found. I have seen this done very often, and in but one instance have I seen it work successfully. It seems to me that these local men frequently have side issues to consider and their positions are often used for their own benefit, largely more than for the interest of the company they profess to represent.

Replying to Mr. Shinn's remarks in regard to filing right of way maps: Had I thought it necessary, I would have expressed myself more fully in regard to this point. My understanding of filing right of way maps is the binding of them in heavy paste-board backs, a little larger than the maps, the maps of each county being bound in a separate atlas. These maps are properly lettered and numbered, indexed in the Map

Record book and filed in one of the map cases, just as are all of the other maps and so forth, of the company. By reference to the index any map or plan can be found at once. The right of way maps for record in counties are bound in the same kind of atlases as are those of the chief engineer's file, except that for the counties the maps consist of blue-prints of the tracings, which prints are taken on linaura, or blue-print cloth.

I note what Mr. Wallace has to say in regard to promises made to land owners by the right of way agent, to which the Chief Engineer finds it difficult to adhere. Just here there occurs to me a settlement of right of way, made by a right of way agent whom I know, and I will give the particulars in regard to it, as they will illustrate a point which I have mentioned. This agent, without any authority whatever, agreed for a side track to be built by the company to a saw-mill which had not been definitely located, as a consideration for a very small parcel of land for right of way. The agreement signed did not specify the width taken for the right of way for the main line and gave no land whatever for the side track, therefore the material for the side track became the property of the land owner as soon as put down. As the mill had not been located it could be placed any distance from the line and a local jury might compel the company to build to it. When a width of 50 feet on each side was prescribed as being a necessary width of right of way for the main line, the land owner looked on the assertion with suspicion and said to the ends of the cross-ties only would be necessary. This is also a roving commission, but with a difference from that mentioned by Mr. Brush, which is, that the company is to do all the paying and the man to whom the side track was promised is to have all the benefits. Again, a consideration of an annual pass is not uncommonly promised for right of way, and the facts of a limit of time for the pass to run, and over what lines it is to be good, do not occur to the unauthorized person making the promise. In such case it is a noticeable fact that the company may object as much as it pleases, but some local jury or legal technicality will make the agreement hold good, and the company will have to make it so. I have known of promises of money to build depots, to give a parcel of land and so forth, to be made to the company if the line is built on a certain route. The railroad company will comply with its part of the agreement, but it rarely gets the consideration promised, and is often afraid to press it's just claim, for fear of becoming unpopular. My experience is that the land owner does not need much pity. He can generally take care of himself very well and does so very successfully.

I think it proper that I should explain myself more clearly as to my expressed opinion of lawyers in right of way matters. I am not averse to them when they are honest men and good lawyers and attend promptly to what is laid before them, and when such lawyers can be found they are very valuable to the engineer, and are always appreciated by him.

But there are bad as well as good lawyers, as there are bad as well as good men in other professions and lines of work, and the railroad companies, although they give up large amounts for legal advice, do not always get those lawyers who properly attend to the company's business. It is men such as these to whom I intended to refer, and not to those who do their duty in good faith.

My paper, good or bad, has caused some discussion, and I hope that the agitation of the subject will not end here, but that several of the members will write their ideas and experiences in right of way matters, so that others who want to learn may profit thereby.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

502.
(Vol. XXV.—September, 1891.)

DISCUSSION OF PAPER No. 478, THE RAVINE DU SUD.

By W. R. HUTTON, M. Am. Soc. C. E.; J. FOSTER CROWELL, M. Am. Soc. C. E.; RUDOLPH HERING, M. Am. Soc. C. E.

WILLIAM R. HUTTON, M. Am. Soc. C. E.—From an inspection of the curves of level on the map it appears that the former bed of the ravine lay on top of the "cone of dejection," continually extending it and consequently reducing its own slope until it overflowed laterally and followed the steepest inclination to the plain. The accompanying profile of the present banks, made from the figured contours, shows a decided reduction of slope below route Torbeck, followed by the steep descent to the plain. Where the steeper slope above meets the flattening at route Torbeck, the stream overflows. The water remaining in the bed cuts for itself a narrow but sufficient channel as it descends the steeper grade. The enlargement of the waterway and the protection of the banks along the more level portion—perhaps even an increase of slope by a moderate excavation of the bed—would prevent overflows at this place. At the foot of the "cone" is still another and greater change of grade. The profile of the new channel shows nearly the same features.

The value of a contraction with reservoir space above in regulating flood discharges, is well known. Although the data in this case are very meager, there is reason to believe that the absence of overflow in the narrower channel below Drouet is due to the steeper slope rather than to any reservoir space in the bed of the torrent. The discharge of

8 000 cubic feet per second (4 000 cubic feet according to Mr. Hering) would fill the bed of this section in a few minutes.

The combination of creosoted timber with the concrete revetment of the banks of the Ravine du Sud would seem to be unnecessary. The concrete alone is sufficient in quantity, and its "indestructibility" is compromised by the timber, which will not last for ever.

Referring to the "bottling" theory—in other words the value of a reservoir as equalizing the discharge of a stream, the following detail may be of interest. The receiving reservoir of the Washington Aqueduct is formed by an earthen embankment about 70 feet high across the valley of Little Falls Branch, a stream of small drainage basin and discharge. The reservoir has a water surface of about 60 acres; the overflow being through a cut in the (soft) rock of a spur near the embankment. As originally designed it was 50 feet wide, the bank rising to 7 feet above the bottom of the cut. In August, 1856, when the construction of the dam was just begun, a severe rainstorm, from 5 to 6½ inches during the twenty-four hours, swelled the stream to the dimensions of a river, and carried away the work already executed.

A computation by the Prony formula—the best in use at that time —from observed flood marks, indicated a discharge of over 5 000 cubic feet per second. Supposing this quantity to flow into the reservoir beginning at its normal level, and to continue at this rate while the outflow over the 50 feet of outlet took place at a rate varying with the constantly increasing head, it was found that in about three hours the surface would have risen 10 feet, when the discharge would about equal the inflow. In consequence the bank was raised to 15 feet above normal water, and the cut widened to 100 feet. Either one of these changes

would have been strictly sufficient, but as a prudent constructor Captain Meigs, the Chief Engineer, adopted both of them.

It is doubtful if Mr. Hering has acted wisely in tabulating the results of the different formulas as given in the discussion of this paper. He certainly shows them a courtesy to which they are not now-a-days entitled. They are of no use to the expert; and they mislead the inexperienced by the inference that they are of some value, although their discrepant results should discourage him. Most of them are mere co-efficients assumed by their "authors" or derived by averaging discordant observations, and are now known to have no conceivable merit, for they take no note of conditions necessary to the solution of the problem. No one knows this better than Mr. Hering, who has done so much to promote the use of exact methods in this particular. These remarks apply to those with constant values of the co-efficient of velocity. Of the others, the formula of Humphreys and Abbot was derived by careful and skillful analysis of numerous observations upon the Mississippi River, compared with reported experiments upon other streams. The value of the comparisons was diminished by the necessity, real or supposed, of applying certain corrections to the observations made by others. However accurate when applied to the river for which it was constructed, experience has shown that this formula is not of general application.

The formula of Darcy and Bazin was a long advance upon those which had preceded it, introducing conditions which had not previously been recognized. It is now replaced by that of Kutter, of which it formed the foundation. The Kutter formula is purely experimental, constructed with the view to represent the results of all accessible and reliable observations, and at this day is conceded to be the only one that is other than by accident correct. These questions have been more fully discussed in Vol. IX, page 217 of our *Transactions*.

On page 480 of the present discussion is a table of twelve rivers on the northern slope of the Alps. The values of the co-efficient of roughness n vary from 0.0294* to 0.0485; the co-efficients of velocity c from 31.9 to 69.7, yet these are summed together and an average value stated for each—for what purpose it is difficult to imagine. If for comparison with the Ravine du Sud, it would have been more in conformity with the principles that have led to the construction and use of the Kutter formula to select one, or a few examples, corresponding as to conditions with the Ravine du Sud, and accept the value of n given by them. The reason for assuming the value of n rather than of c is not sound; c is to be computed from definite data, the slope, the mean depth and the value of n. Only the last is a matter of observation and judgment, and that within narrow limits. It is to be obtained by com-

* This is a correction of a printed error on page 480.

paring the local conditions as to roughness of bed with that of other streams for which the co-efficient n has been obtained by experiment.

Mr. Crowell did not expect much from the formula used, Stevenson's, which, however, answered his purpose of comparing discharge in the typical section with the discharge in the new, despite the remarkable errors pointed out by Mr. Hering. As he has attributed to the new channel a third more carrying capacity than to the typical section, accuracy, either relative or absolute, was not necessary.

J. Foster Crowell, M. Am. Soc. C. E.—In reply to Mr. Hutton, the author of the paper would say: A short distance below the route Torbeck, opposite station 1800 (meters), and indicated on the map accompanying the original paper (see Plate LV), are two small houses, one of which is a one-story frame structure covered on the outside with lath and plaster. This little building stood through the great flood which carried away the protective works at station 800. The outside plaster was washed off by the water at that time, according to eye-witnesses, to a height of about 3 feet above the ground, and the remaining plaster thus gives a well-marked water-line, which was probably the maximum of the flood. This fact would indicate that the overflow below Torbeck, referred to by Mr. Hutton, was due to the insufficiency of the water-way further down stream and not to the flatter slope here. If there had been any considerable velocity in the flow, the frail building would have been carried away. The water doubtless spread slowly out, forming a lake, while the flood was rushing out through the gap it had made in the revetment, 1 kilometer further up stream. The contours show that the old bed near where the break occurred is not on the highest part of the "cone of dejection," which would, on the contrary, appear to be on the other side (the western side) of the present bed.

But whatever the immediate cause, the original plan comprehends all of the three features which Mr. Hutton suggests, viz.: "the enlargement of the water-way, the protection of vulnerable portions of the banks and an increase of slope by a moderate excavation of the bed," the latter being possible by adopting for the bottom of the new channel a uniform gradient of $26\frac{5}{10}$ feet per mile, from about station 700 to station 2200 (p. 477), as against 17 feet to the mile, which is now the slope through the portion of the stream below Drouet not overflowed, and where the typical section was measured. As to the cause of the absence of overflow at the latter point, Mr. Hutton is undoubtedly correct in his view that it is the steeper slope relative to the compartment just above, which has been described as the "reservoir compartment." This does not, however, conflict with the view expressed in the paper as to the effect of the reservoir space above the route Drouet (pages 473 and 476), in reducing the original velocity of the torrent and storing up the accumulating flood water so as to give time for the "neck of the bottle" to empty it.

The use of creosoted timber in connection with the concrete revetment, which Mr. Hutton criticises as unnecessary, is proposed as a means of obtaining immediate security for the concrete walls against concentrated scour, in the event of a flood during the progress of construction, and lateral stability until such time as the embankment backing can be fully completed and made efficient. The formation at the most vulnerable points consists of a thin covering of clay overlying a bed of sand, requiring a very deep foundation if concrete alone were to be used. Ultimate reliance for the stability of the wall can be obtained from the heavy embankment of small stone and clay, which, as it will be formed of surplus material from the channel excavations, will add little to the cost. The timber below the plane of saturation is designed for a permanent foundation. The extension of the timber members above that plane is for the purpose of temporarily giving the required stability in case of a flood intervening during construction, and it is so disposed as to permit of its gradual decay without impairing the integrity of the concrete wall or the necessity of renewal. By the use of the timber a very considerable saving in the quantity of concrete can be effected; it also permits of the forming of the embankment backing, either in advance of, during, or after the readiness of the concrete to retain it, a very important consideration in a work so remote from the source of supplies.

A word as to Mr. Hering's action in tabulating the results of the different formulas may not be inappropriate here, for it was at the writer's express solicitation, and perhaps only on that account, that he contributed it to the discussion. It appears that he has brought out one of the most important points in the discussion, by making clear to those of us whose special training does not fit us to readily discriminate as to the relative worths of different recognized flowage formulas, the essential points of difference between the true and the false; a result to which Mr. Hutton has also contributed by his comments on the tabulation.

Rudolph Hering, M. Am. Soc. C. E.—Mr. Hutton has called attention to the fact of my having averaged the results of certain river gaugings, and very properly takes exception to applying this method in a general way. It is curious that I should have exposed myself to this criticism when I have always strongly opposed such averages, when it was one of the criticisms I made myself upon Kutter's original table, and when I took such pains to compile and have printed in the translation of Kutter's work (made by Mr. Trautwine and myself) all the many individual gaugings, instead of Mr. Kutter's averages for them. I quite agree with Mr. Hutton on this question. My object in giving, besides the individual values in Table No. 2, also the average "in round numbers," was simply to have a single quantity with which to show the effect of the different formulas still occasionally quoted, and particularly Mr. Crowell's modification of Stevenson's formula. The latter was the sole

object of my discussion rather than to suggest the best co-efficient for the Ravine du Sud. The average assumed by me—sufficiently near for the above purpose—was confined to those streams whose beds, according to the descriptions, were approximately like that of the "Ravine," and it also agreed almost exactly with the values obtained from the two streams, "Isar" and "Saare," which most nearly resembled it in slope and in mean radius.

In reply to Mr. Crowell, who thinks my comparison is not fair to the Stevenson formula, because I do not represent it "in the original form," it is right to point out that, while in Table No. 1 the eighth formula is Mr. Crowell's modification, the sixth is the original Stevenson formula "for large streams," as contained in Humphreys' and Abbots' report on the Mississippi River, page 220. It is true that Mr. Stevenson also proposes a co-efficient $c = 69$ (when the formula is reduced to feet and seconds), but this is intended only "for small streams under 2 000 cubic feet per minute," or such as have less than 1 per cent. of the probable discharging capacity of the Ravine du Sud! It is this smaller co-efficient reduced to other units of measure which Mr. Crowell applies to the Ravine du Sud, and finds the results to correspond closely to those of the Kutter formula. Surely he does not mean that such is more fair to the Stevenson formula.

In conclusion, I wish to remove a possible impression to the effect that I was criticising Mr. Crowell's ultimate designs unfavorably. Such was not my purpose. The manner in which he has applied Stevenson's formula (see pages 476 and 477) eliminates errors in his case. He assumes a hypothetical flood, finds from the measurable elements a hypothetical discharge, and uses the latter simply to ascertain for the same channel a new cross section for a different slope. In this operation the hypothetical discharge cancels, as does also the co-efficient c, when constant. Therefore, it is entirely immaterial what the values are, either of the discharge or of the co-efficient c.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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503.
(Vol. XXIV.—October, 1891.)

THE AMERICAN RAILROAD VIADUCT.—ITS ORIGIN AND EVOLUTION.

By J. E. GREINER, M. Am. Soc. C. E.

WITH DISCUSSION.

A new era in the design of a certain type of bridge began in 1868, with the construction of the Bullock Pen Viaduct on the Cincinnati and Louisville Short Line Railroad, "now L. and N.," and the history of the American Railroad Viaduct is mainly a statement of its development since that time.

From the beginning of railroads, engineers have been continually directing their efforts to produce structures which would not only fulfill all requirements, but which at the same time might be superior and bolder than any before; consequently each new form of truss, column or detail gave to the profession valuable suggestions of what could yet be done in so fertile a field for inventive ingenuity. These suggestions naturally formed the nucleus from which new ideas were evolved, until by the continued addition of improvements and the elimination of palpable fallacies of design, the present standard type of single quadrangular and the later, more economical, but not quite so simple, Pegram trusses, were gradually produced.

A well known engineer has said that "it requires a great deal of nerve for any one to say that he originated a certain type of bridge structure or a specification, when it all came about by some one adding a little to what some one had already done," and the viaduct is no excep-

tion, since it was not of spontaneous production, neither was its conception quietly nurtured in the brain of some engineer, from whence it sprang upon public notice fully matured. It followed a very gradual course of development, indications or suggestions of certain features being distinctly traced to the earliest timber trestles. The general plan of these trestles was practically the same as is now used, being formed of bents stationed at short intervals, connected together at their tops and at intermediate joints by longitudinal lines of stringers or struts. Each bent consisted of two or four legs with cap and sill, all cross-braced, the outer legs having a batter. An outline sketch of such trestles is shown on Plate LXXVIII, Fig. 1. Of course as the depths of ravines or depressions to be crossed increased, the general character of trestles was modified somewhat, the most striking example being the old Portage Viaduct, a structure 800 feet long and 234 feet high from bed of river, designed by Silas Seymour in 1851, built in 1852, and completely destroyed by fire in 1875. This viaduct was always considered the boldest attempt ever made in timber trestles.

The first notable change made in the established wooden trestle, was in the Carey street trestle over the Baltimore and Ohio Railroad tracks in Baltimore, built by Wendal Bollman, in 1851. Here for the first time the successive bents were held together longitudinally by iron diagonal rods connecting the top of one leg to the bottom of another. These rods took hold of pins passing through cast shoes at the bottom of the posts, and at the top were provided with screw ends and nuts which bore against cast washers on the upper surface of the chord pieces or stringers. The bents were in two stories and consisted of two legs, cap, sill and intermediate strut, cross-braced, all parts being of wood except the longitudinal tie-rods and cast shoes. A section of this structure is shown in Plate LXXVIII, Fig. 2. The old Antietam trestle on the Washington County Branch of the Baltimore and Ohio Railroad was of similar construction. It was, however, two stories high throughout.

Since "the first impulse to the general adoption of iron for railroad bridges was given by Mr. Benjamin H. Latrobe," it is not surprising that the first iron trestles were designed by his assistant, Mr. Albert Fink, Past President Am. Soc. C. E., built by the Baltimore and Ohio Railroad Company in 1852, and erected early in the year 1853. These were known as the Buckeye and Tray Run viaducts. They crossed ravines on Cheat River, the former being 350 feet long, 46 feet high from top of wall, and

28 feet wide between hand-railing, and the latter 445 feet long, 58 feet high and 28 feet wide. All parts of the trestles were of cast-iron except the tie-rods, the general design being shown on Plate LXXVIII, Fig. 3. These structures rested on continuous walls of masonry, and having been built in sections of about 125 feet in length, could expand and contract with changes of temperature. It may be interesting to learn that it was the original intention to build the walls to full height, but after having reached a certain elevation, the engineer considered it not advisable to continue, owing to evidences of weakness, and as a result of this decision, Mr. Fink designed the structures here described. The Buckeye was removed in the year 1884, and the walls, after being properly strengthened, were built up as originally intended. The Tray Run was replaced by a modern structure in 1887.

In 1852, a tower lighthouse at Cary's Fort Reef was designed. This was composed of four columns, one at each angle, connected together in all directions by struts and rods in the same general manner as are the towers of viaducts built at the present time. The tower was of two or more stories, and the posts or legs had a batter both longitudinally and transversely, instead of being battered in a transverse direction only, as is now adopted for viaducts. The lighthouses at Sand Key designed in 1853, Sombrero Key (Dry Dock 1857), Shell Keys and Alchapalaya in 1859, were of similar construction.

The Crumlin Viaduct, begun in 1853 and completed in 1857, on the Taff Vale Extension Railway, South Wales, Messrs. Liddell and Gordon as engineers, is in all probability the first iron viaduct resting on independent iron towers. The main part of the viaduct consists of seven spans of wrought iron Warren girders 150 feet center to center, and six towers arranged in tiers, each containing fourteen cast-iron columns. The greatest height above the surface of the water is about 200 feet. A section of this viaduct is shown on Plate LXXVIII, Fig. 4.

On Plate LXXVIII, Fig. 5, is an outline sketch of the iron structure over Jordan Creek, on the Catasauqua and Fogelsville Railroad, designed and built by the late Mr. F. C. Lowthorp, F. Am. Soc. C. E., in 1856 and 1857. The total length was 1 122 feet, consisting of nine spans of 100 feet each, and two spans of 111 feet each, supported on cast-iron trestle towers about 89 feet high. All compression members were of cast-iron, and tension members of wrought-iron rods, capable of adjustment. In 1889 the Catasauqua and Fogelsville Railroad were hauling cars

weighing, when loaded, 88 500 pounds, and not feeling safe with such heavy loads on the old bridge, they had it replaced by a more modern structure, resting, however, on the original masonry erected by Mr. Lowthorp, the only change being three new coping stones.

From 1857 to 1868 there were no material improvements made in the general design of viaducts, superior to those embodied in the Crumlin and Jordan River structures; and in these cases, it will be noticed, the towers were grouped together on top of solid masonry piers. These piers being massive, required great care in preparing safe foundations in unfavorable situations; besides, the necessary large outlay for such structures induced many railroads to adopt cheap wooden trestling in preference. Engineers, therefore, began to study more closely the principles of true engineering in relation to iron viaducts, and directed their attention toward designing a system of iron piers which should be of cheap construction, adapted to all conditions, and free from inherent defects.

The first evidence of the result of such investigations is seen in the Bullock Pen Viaduct, designed by Smith, Latrobe & Co. in 1867, built in the same year, and erected on the Cincinnati and Louisville Short Line, now "L. and N.," in 1868. The structure was 470 feet long, the maximum height 60 feet, and of the same general design as the viaduct Plate LXXIX, Fig. 1. Mr. Frederick H. Smith, M. Am. Soc. C. E., a member of the firm of Smith, Latrobe & Co., personally made the plan, although there were suggestions from, and consultations with, both C. Shaler Smith, M. Am. Soc. C. E., and C. H. Latrobe, M. Am. Soc. C. E. Mr. F. H. Smith, however, considered that he originated certain features entirely independent of any other assistance, claiming as new and original with himself the following, which form part of his specifications in Letters Patent, No. 89 948, filed at Washington on October 13, 1868, and issued May 11, 1869:

"*First.*—Anchoring the tops of iron or steel piers of bridges by means of longitudinal diagonal stays, or equivalent tension members, to the feet or pedestals of other piers or abutments, so that the pier can be formed of but two columns, and still have the great longitudinal base afforded by the spread of the longitudinal diagonal stays to give it stability under the action of temperature or moving load.

"*Second.*—In combination with the foregoing system of stays, iron or steel columns, stiffened by means of up-and-down struts or tension-rods and cross-struts or tension-rods at such point in the height of the columns as will prevent flexure.

"*Third.*—Stiffening iron or steel bridge piers by means of longitudinal rods or struts, anchored to or abutting against masonry, and connecting therewith the piers, and the piers with other piers, at such points in their heights as will prevent flexure.

"*Fourth.*—The combination of iron or steel piers of bridges with the longitudinal diagonal stays, and up-and-down stiffening system and the longitudinal stiffening system."

The firm was building five other viaducts about the same time for the same road, all of the general style of the Bullock Pen.

The Running Water Viaduct, on the Nashville and Chattanooga Railroad, erected in 1869, and the Lyon Brook Viaduct on the New York and Oswego Midland Railroad, Plate LXXIX, Fig. 2, built by the same company about the same time, were similar in design, the former consisting of one Fink span of 95 and 692 feet of trestling, the maximum height being 115 feet, and the latter of one span 100 feet long and 720 feet of trestling, the maximum height being 155 feet. In these two structures the longitudinal diagonal bracing and timber strutting do not extend from abutment to abutment, as was the case in C. and L. S. L. trestles, but the ends of the long channel spans rest upon trestle-bents which are connected to adjacent bents by wrought-iron struts and tie-rods, thereby forming complete iron towers of two bents each, with a 30-foot tower span between the bents. In these structures there appears for the first time the common tower, composed of two bents of two legs each, braced together in all directions, the legs having a transverse batter only, and the bents being connected at their tops by a free or swinging span. This is the typical braced tower as used in American viaducts, and its use in these cases was not due to originality of conception so much as to the necessity of the cases, which demanded a pier stable in itself at each end of the long open span, for the continuous lines of longitudinal strutting to bear against ; since at that time it was thought essential to continue such strutting from abutment to abutment. By using these braced towers the structure was practically composed of two trestles, each of which was complete in itself, but connected by an open span so as to form one whole.

The features in the foregoing trestles, which were original with C. Shaler Smith and C. H. Latrobe, can be briefly stated as follows :

First.—Cast-iron foot-boxes interposed between and forming the bearings for the column sections, in combination with longitudinal

diagonal ties connecting the joints of any one tier with those of another tier. *Second.*—The construction of the cast-iron postcaps, foot-boxes and joint-boxes. *Third.*—The combination of a series of independent or isolated bridge piers, with an intermediate span or spans of truss connecting the same. *Fourth.*—Yoke and screw end attachment for longitudinal diagonal ties.

Messrs. Smith, Latrobe & Smith, up to this time (1869), believed that it was quite necessary to make the continuous longitudinal struts of wood in order to prevent the contraction and expansion which would follow the use of a similar system of iron struts; but in the design of the towers at the ends of the channel-span in the before-mentioned Running Water and Lyon Brook trestles, Mr. C. Shaler Smith conceived a means of doing away with wooden struts and substituting iron in their place, and in a general design of a viaduct made in 1870 he contemplated the use of iron throughout, and provided for expansion and contraction by omitting all bracing in certain panels or bays, thus forming a structure "made up of fixed piers composed of any number of panels of trestles alternating with spans of truss having one or both ends loose, so as to permit of contraction or expansion." Plate LXXX, Fig. 1, is a copy of this general design which accompanied his application for the patent, No. 111 010, granted January 17th, 1871.

The Verrugas Viaduct, on the Lima and Oroya Railroad in Peru, built in 1871 by the Baltimore Bridge Company, and designed by Associate Engineer C. H. Latrobe personally, was the first wrought-iron structure built by the company embodying the principle of independent braced piers, as set forth in Mr. C. S. Smith's specifications. This structure consisted of three towers or piers and four Fink trusses, the total length being 575 feet. The central pier was 252 feet high, and the others 146 and 179 feet. Each tower contained three bents, and each bent composed of four columns braced together in all directions. A full description and cause of collapse, in 1889, may be found in the *Engineering News* of July 27th, 1889, and December 27th, 1890, and also in the *Engineering and Mining Journal* of January 24th, 1891.

These structures as built by the Baltimore Bridge Company, became very popular. In less than five years after the Bullock Pen Viaduct was erected, the company had built over 15 000 lineal feet of railroad viaducts, and since letters patent were granted in England, France and Belgium, as well as in the United States, we are safe in presuming that such structures were originated in this country.

In 1869, Mr. Thomas C. Clarke, M. Am. Soc. C. E., designed a viaduct approach for the proposed bridge across Blackwell's Island, New York. (Plate LXXXI.) In a recent paper read before the American Society of Mechanical Engineers, he states that at that time, not wishing to use any perishable material like wood, the difficulty arising from changes of length due to changes in temperature was avoided, by simply leaving out the longitudinal bracing and struts of each alternate bay, thereby giving a series of braced towers 30 feet long, connected by girders of the same length resting on their tops, and having a sliding motion at one end. He afterward found it expedient to lengthen these intermediate girder spans to the greatest length that could be transported from the shops to the site, about 60 feet, and believes himself to be the originator of the well-known American Railroad Viaduct. This seems to be true, since, although Mr. C. S. Smith made application for letters patent on a similar general design in 1870, Mr. Clarke had anticipated him a whole year.

About the same time (1870) Mr. C. Bender secured letters patent on the type of viaduct shown on Plate LXXXII, Fig. 1, and modestly claimed that he could build such a bridge "cheaper and more perfect than any other one in existence." He seems to have gotten the idea of his design from an ordinary kitchen table, and refers to the spans with overhanging ends as "table spans." This system differs from those previously designed, by combining the piers or trestles with the spans themselves, thereby using the stiffness of the truss itself against flexure, as the element of stability, "likewise as the frame of the common table keeps in position the table legs." If any such structure has ever been built in this country, the author has been unable to ascertain when or where. This invention is mentioned here not because it in any way contributed to the evolution of the typical viaduct, but merely as showing the general aim of engineers at this time.

In 1871 Mr. Edward Serrell, of New York, invented some improvements in iron viaducts and claimed as his invention a "metallic tower made of columns around a central column that is permanently attached to the foundation, and the surrounding columns are provided with foot plates in shoes that are attached to the foundation so as to allow for expansion and contraction, but to hold the parts properly in position." Details of this arrangement are shown on Plate LXXXII, Fig. 2. The most interesting part of Mr. Serrell's design is his method of providing for

expansion, as can be seen from Fig. 2^a, where the arrows indicate the direction of motion, and Fig. 2^b the method of keeping this motion in a straight line. The principle shown in the sketches is now quite commonly used at the sliding ends of plate girders and stringers; and the same could be applied with great advantage to the alignment of viaducts, if in a four-legged tower one column be permanently fixed and the others be allowed to move in the directions shown by arrows in Fig. 2^a. The author does not know whether the application of the principle of sliding motion shown in Plate IV, Fig. 2^a, has ever been used on viaducts, and believes that the double motion obtained by the arrangement shown is new, although there are other means of providing for sliding of legs, all more or less satisfactory.

In 1875, Mr. G. Bouscaren, M. Am. Soc. C. E., built his first viaduct on the Cincinnati Southern Railroad. This is known as the Horse Shoe Run Viaduct* (Plate LXXX, Fig. 4), and was made in imitation of similar structures built by the Baltimore Bridge Company. Mr. Bouscaren consulted with Mr. C. S. Smith at the time, and the latter still considered it essential to have a continuous line of diagonal rods from the top of the first to the feet of the last trestle bent in a group, thus making the number of bents in any particular group, dependent on the height. In 1876 Mr. Bouscaren adopted braced piers composed of two trestle bents only, braced together in all directions, no matter what was the height of the structure. The foot of one post was fixed to the masonry of the pedestal, and the other three were allowed to move on friction plates with the changes of temperature. This was a very marked improvement on his previous types, inasmuch as he avoided the necessity of frequent readjustment of diagonal bracing-rods. Fishing Creek Viaduct (Plate LXXX, Fig. 5), built in 1876, is of this type. In 1878 an additional advance in the economy of design was made by proportioning the length of span between piers to the height of the structure; McKee's Branch Viaduct (Plate LXXX, Fig. 6) is an example of this type.† In 1881 the same engineer designed for the same road the first viaduct on rocker piers, made of a single trestle hinged at both ends. Mr. Bouscaren considers that this type is far superior to all others when circumstances will permit of its use. The transmission of stresses is more direct (no adjustment whatever is necessary), and they are more

* Designed and constructed by the Keystone Bridge Co. See Appendix.

† Designed by C. L. Strobel, and approved by G. Bouscaren, Chief Engineer.

uniform and better defined than in any previously designed structures. The pin-bearing shoes, at the feet of the trestle posts on one side, are fixed to the masonry pedestals, and those on the other side are free to move on friction plates.

A great number of viaducts copied from those designed by C. S. Smith and G. Bouscaren, have been built, but the greatest height of towers yet attained seems to have been in the Kinzua Viaduct, where the greatest height of any tower is 297 feet, or from base of rail 301 feet. The viaduct was built by Clarke, Reeves & Co., 1882. Drawings and full description of the same may be seen in the Album of Designs issued by the Phœnix Bridge Company.

The new Portage Viaduct (Plate LXXX, Fig. 3), G. S. Morison, M. Am. Soc. C. E., engineer, built in 1875, if not the very first, seems to be the first of particular note which had towers of two bents only for the different lengths of spans. This structure replaced the old wooden viaduct, some of the original piers being used. These piers being 50 feet, center to center, the designer could have made all spans 50 feet, but considered the adopted arrangement preferable. This type of structure therefore antedates by several years similar types designed by G. Bouscaren for the Cincinnati Southern Railroad.

Since the year 1878, general designs have been practically uniform, embodying the distinctive features which make such structures the most economical; but the Kinzua, so often spoken of and illustrated as a striking example, still holds its own against all comers for best adaptation of design to existing circumstances.

The question has been often asked, "Who was the originator of the American viaduct?" It is now time that the question be settled once for all, and there is no society better able to do so than the American Society of Civil Engineers, which includes among its members many who were important factors in the early progress of bridge engineering.

In the foregoing history the facts given are perfectly authentic, but there may be other structures known to some, which could be considered as additional links in the chain of evolution. Be this as it may, there are ample examples given for any one to trace the gradual development of viaducts from the early timber trestles; for in these structures we see the bents composed of two or four legs, and the outer legs battered, built in stories and cross-braced, a very similar construction to the bents of later viaducts in iron. The second step is seen in the old Carey Street

trestle built by Bollman in 1851, where he anchored the tops of bents to the feet of adjacent bents, or, in other words, introduced a continuous system of longitudinal diagonal bracing. Then came the complete iron bents in the Tray Run and Buckeye trestles; next the towers composed of four columns, arranged in tiers, braced in all directions; and finally the Crumlin and Jordan Creek viaducts, in which the different spans rested upon built up iron towers. This is the end of the first stage of development, and includes all features up to 1867.

If, then, up to this time we find bents, cross-bracing, longitudinal bracing and towers, what was novel in the Bullock Pen trestle? Merely the combination of some of these features, and their adaptation to wrought-iron construction. As said before, this structure was the beginning of a new era, and it must be acknowledged that Mr. F. H. Smith, not C. Shaler Smith, was the actual designer and should have full credit in introducing a new combination, therefore a practically new style of structure, in wrought-iron. Mr. C. S. Smith, however, was closely connected with this work, and the trestle has a feature, namely, the connection of the longitudinal diagonal bracing with intermediate joints on the columns, which was suggested by him. F. H. Smith introduced the continuous line of timber strutting which anchored the bents to masonry abutments. In the Running Water and Lyon Brook structures, the distinctive feature is the complete iron tower composed of two bents of four legs each, braced in all directions by iron rods and iron struts, besides having a regular span to connect the bents which form a tower, originated by C. S. Smith.

Finally we come to the structure composed of towers with spans connecting the bents of the same, alternating with free or open spans. Mr. Clarke justly claims to be the designer of this, the typical viaduct, and the facts seem to uphold his claim. While it is true that Mr. C. S. Smith patented such a structure, it is a well known fact that none of the patents taken out by members of the Baltimore Bridge Company could stand the tests of a court trial in view of what had been done before, and they soon became public property. The first structure embodying the principles of such a type was the Verrugas, built by the Baltimore Bridge Company in 1871. It is true that it embodies certain ideas concerning the number of bents in a tower which finally came into disuse, but nevertheless it can surely be considered typical.

It was the original intention to embrace in this paper a discussion of

construction, details, etc., also to develop formulas for weights of viaducts, but the historical portion having consumed considerably more time than was anticipated, the author finds it necessary to close for the present, trusting that others better qualified will at some early date present the Society with a paper on viaduct construction, a subject which has heretofore received but a small part of the consideration to which it is entitled.

In conclusion, the author would like to call attention to the loose way in which the terms "trestle" and "viaduct" are applied, even by well-known and intelligent engineers. Of course we all know that the usual definition of a viaduct is "a bridge of any style which carries a road over a ravine or depression where there is little or no water." A definition of a trestle would be practically the same with a limit to the style, although we feel instinctively, as it were, that the term "trestle," applied to the Kinzua Viaduct, would be a misnomer. Where, then, does the trestle end and the viaduct begin?

Further, suppose we have two profiles which would admit the use of the same structure. In one case there would be no water, but in the other, water would cover most of the depression. Should we call one of these structures a viaduct and the other simply a bridge, or should they both be viaducts? The derivation of the word will not help us, and it seems preferable that the name should designate the style rather than the location. In this sense a trestle would be a bridge of wood or metal in which the different spans are supported directly upon trestle-bents not connected in groups or towers; and a viaduct, a bridge of wood or metal in which the different spans are supported directly upon legs of towers composed of two or more bents braced together in all directions. The above definitions, while not meeting English practice, will undoubtedly distinguish American structures, and if adopted, would do away with all looseness in this connection.

DISCUSSION.

GEORGE S. MORISON, M. Am. Soc. C. E.—Reference has been made in this paper to the new Portage Viaduct. I ought to state that although this was designed by myself and built under my direction, the particular arrangement of towers used was fixed, first, by the piers of the old wooden viaduct, and, secondly, as suggested by others. If, as the writer says, this was the first structure of its class, the credit for any originality in this way does not belong to any one person. I have thought that it might be interesting to compare this Portage Viaduct with another viaduct built by myself almost exactly ten years later, namely, the Marent Gulch Viaduct on the Northern Pacific Railroad, the height of the iron work of which differs less than a foot from the height of the iron work of the Portage Viaduct, and the length of which is very nearly the same. The Marent Gulch Viaduct is, however, a symmetrical structure, which the Portage Viaduct is not. As I shall soon present a paper describing this interesting work in detail, I will reserve further remarks until then. I would say now, however, that this structure is built entirely of iron.

G. BOUSCAREN, M. Am. Soc. C. E.—For the purpose of comparing the relative value of this plan and other plans, it would be very interesting to know the weight of iron in the total viaduct and the area in square feet of crossing between the grade line and ground line. The comparison of these data is the best way of arriving at the relative economy of the different plans. At the time when I made the modifications of the plans on the Cincinnati Southern, it was such a comparison that led me to adopt longer spans. Where the masonry foundation had been put in for 30-foot spans, I found it cheaper to leave the short spans out and substitute 60-foot spans instead. The result I arrived at was, that the most economical length of span was about one-half the height of support. The relative lengths of span and piers here conform with that rule for the center, but the spans would be a little too long for the ends.

ROBERT MOORE, M. Am. Soc. C. E.—It strikes me the effort made by the author to confine the use of the word viaduct is open to criticism. Under the limitation proposed I do not know what we should do with the old fashioned stone viaduct, which would then be without a name. I doubt much whether it is practicable to limit the word in that way. I think the etymological use of the word is the best. According to this a viaduct is a structure which carries a road, whilst an aqueduct is one which carries a canal. The suggestion that the credit of the invention of the modern railroad viaduct should be given where it belongs is a very good one, yet I presume it will be found impossible to ascribe its origin to any one person. Probably every viaduct embraces ideas due to many persons, so that I question whether it is practicable to

PLATE LXXVIII
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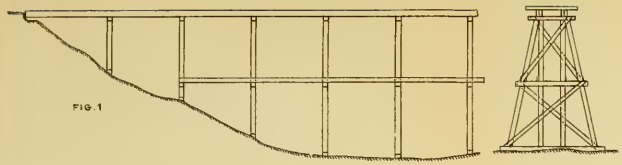
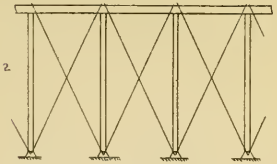
GREINER ON
THE ORIGIN OF THE R.R. VIADUCT

FIG. 1



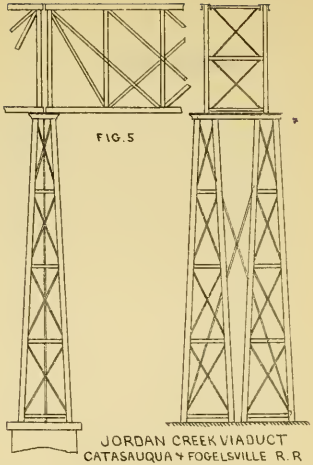
COMMON TIMBER TRESTLE

FIG. 2



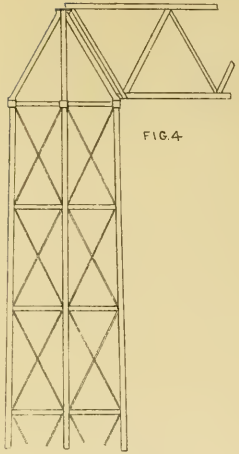
OLD CAREY ST. TRESTLE - BALTº Mº
1851

FIG. 5



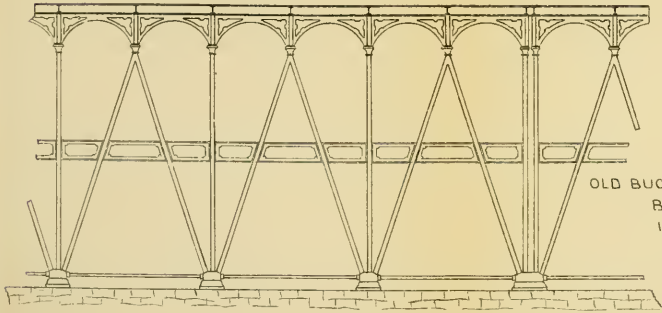
JORDAN CREEK VIADUCT
CATASAUQUA & FOGELSVILLE R.R
1856-7

FIG. 4

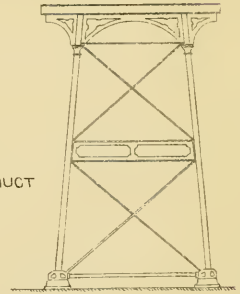


CRUMLIN VIADUCT
SOUTH WALES
1853-7

FIG. 3



OLD BUCKEYE VIADUCT
B & O. R. R.
1852-3



ORIGIN AND EVOLUTION OF THE AMERICAN RAILROAD VIADUCT.

PLATE LXXIX
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GREINER ON
THE ORIGIN OF THE R.R. VIADUCT.

STYLE OF BULLOCK PEN TRESTLE - C. & L. S. L. R. R. - 1867-8

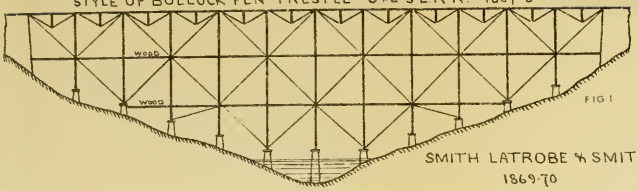
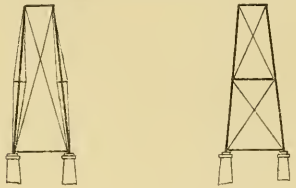


FIG 1

SMITH LATROBE & SMITH
1869-70

ORIGIN AND EVOLUTION
OF THE
AMERICAN RAILROAD VIADUCT.



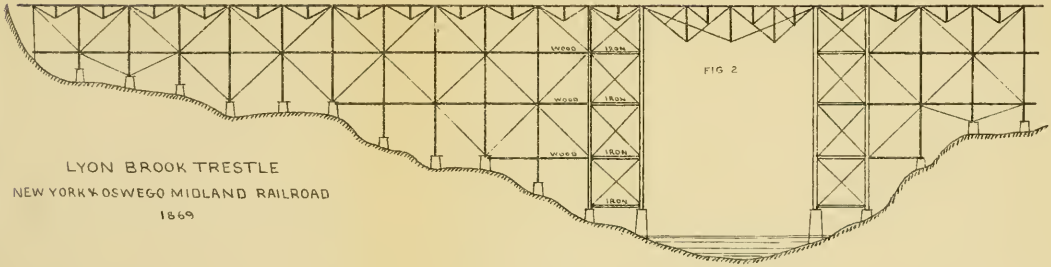


FIG 2

LYON BROOK TRESTLE
NEW YORK & OSWEGO MIDLAND RAILROAD
1869

PLATE LXXX
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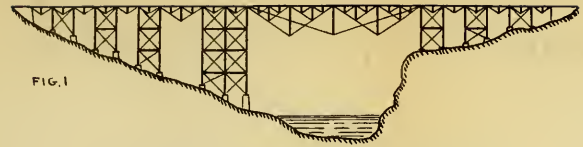

FIG. 1

GENERAL DESIGN 1870 - C. SHALER SMITH.
OLD VERRUGAS VIADUCT-OF THIS TYPE-1871

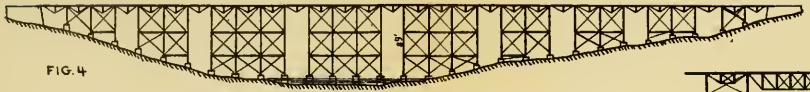

FIG. 4

HORSE SHOE RUN C.S.R.R. 1875 - G. BOUSCAREN

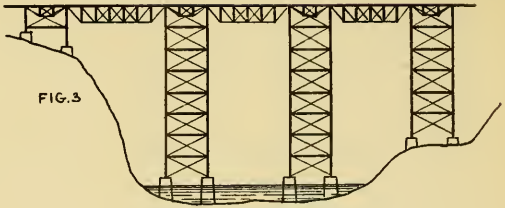

FIG. 3

NEW PORTAGE VIADUCT
GEO. S. MORISON - 1875

FIG. 5

FISHING CREEK C.S.R.R. 1876 - G. BOUSCAREN
DESIGN OF APPR. VIA. OF BLACKWELL'S ISLAND BR. OF THIS TYPE - 1870 - THOS C. CLARKE

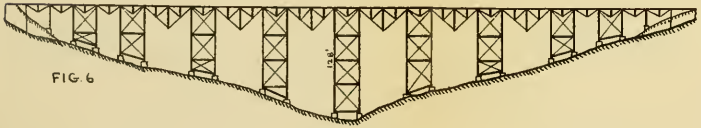

FIG. 6

McKEES BRANCH C.S.R.R. 1878 - G. BOUSCAREN.

ORIGIN AND EVOLUTION OF THE AMERICAN RAILROAD VIADUCT.

PLATE LXXXI.
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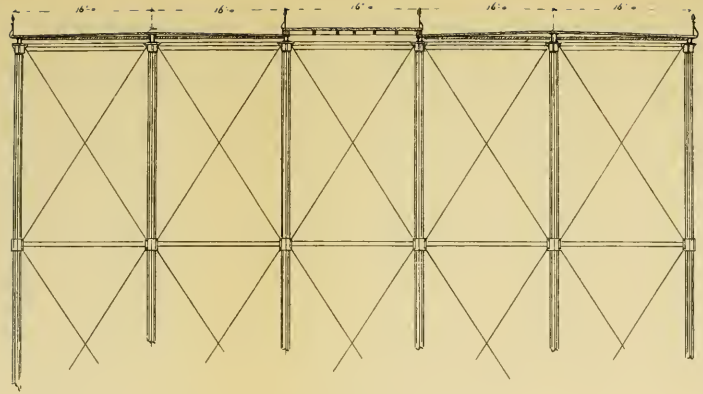
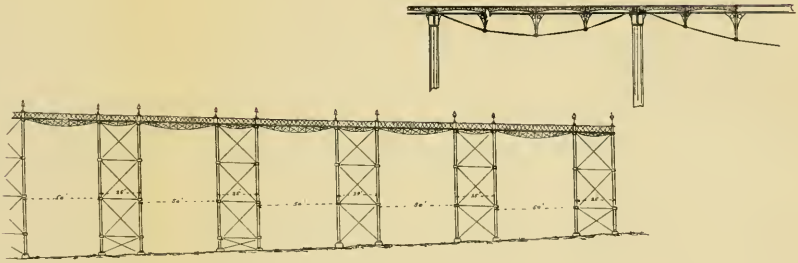
THE ORIGIN OF THE R.R. VIADUCT.



— CROSS SECTION —



Designed by Thomas C. Clarke
Brooklyn, N.Y.
1869.

— ELEVATION —

— SKETCH OF VIADUCT APPROACHES —
— EAST RIVER BRIDGE —
CROSSING BLACKWELL'S ISLAND.

PLATE LXXXI
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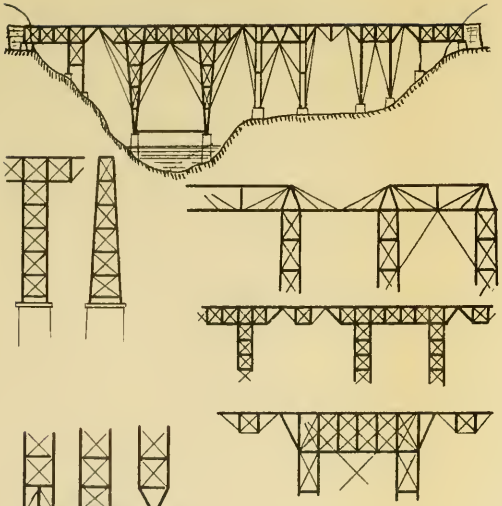
GREINER ON
THE ORIGIN OF THE R.R. VIADUCT.

C. BENDER - 1870



FIGS. 1

ORIGIN AND EVOLUTION OF THE AMERICAN RAILROAD VIADUCT.

EDWARD W. SERRELL - 1871

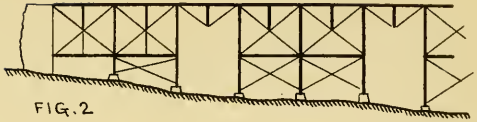


FIG. 2

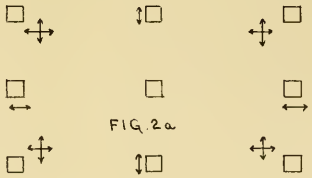


FIG. 2a

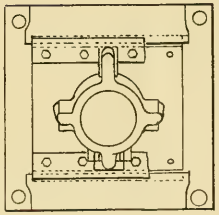
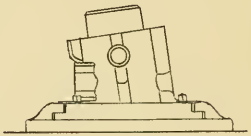


FIG. 2b



attribute to any one person the origin of what is called the American viaduct.

E. THACHER, M. Am. Soc. C. E.—During the past two years I have designed between 10 000 and 11 000 linear feet of iron or steel viaduct; all of it, with the exception of the Hill City approach to the Chattanooga bridge, was for railways, and manufactured and erected by the Shiffler Bridge Company of Pittsburgh. All of this work consisted of iron or steel plate girders spaced 9 or 10 feet between centers, supported by iron bents braced laterally and longitudinally in the form of towers, as has been the general practice for some years. Among the highest of these structures is Buffalo Trace on the Kentucky Midland Railroad, 153 feet; Neal's Branch on R. N. I. & B. R. R., 190 feet; and Marble Creek on the same road, 210 feet in height. This latter structure had some interesting features. The line passed along the edge of a narrow precipice, and the cross-sections were very irregular. In one bent one leg was 52 feet longer than the opposite one, the base and one intermediate point being strutted and anchored back to the rock. In another bent one leg was 26 feet longer than the opposite one. The length of spans in these structures ranges from 36 to 70 feet. Some years ago I made some estimates to determine the most economic span for different heights, and found, using the specifications of the Keystone Bridge Company of that date, and disregarding the cost of pedestal masonry, that for all iron structures the spans most economic of material are as follows:

For heights up to 55 feet, all spans 30 feet.

For heights from 55 to 91 feet, 30 feet towers and 40 feet intermediate.

For heights from 91 to 129 feet, 30 feet towers and 50 feet intermediate.

For heights from 129 to 200 feet, 30 feet towers and 60 feet intermediate.

Using steel girders and taking into account the cost of masonry, the economic length will be somewhat greater than as given above. In Marble Creek viaduct 70 feet spans were used at the highest point. The length of span is governed in many cases by the nature of the foundations and the convenience in erection. When it much exceeds 70 feet it becomes a difficult matter to construct a traveler that will handle the material economically, and unless the structure exceeds 250 feet in height I doubt if anything is gained by the use of spans longer than 70 feet. The statement of the author that the single leg type of structure is cheaper than any other is not borne out very well by the facts. I made an estimate some years ago on a design of that kind furnished, I believe, by C. Shaler Smith, and the cost, as nearly as I recollect, was about double what it would have been with braced bents. The Bullock Pen and other early viaducts on the L. and C. Short Line

Railroad had one defect probably not contemplated by the designer. The inspector who came around often to tighten up the rods with a 3-foot wrench was too ambitious, and succeeded in several instances in lifting the legs off of their pedestals.

The evolution of the viaduct during the past twenty years has been no less in general design than in detail, but as the author does not touch that part of the subject, I will not take it up.

T. C. Clarke, M. Am. Soc. C. E.—My only claim is that I designed the viaduct illustrated in Plate LXXX, for the reasons given by the author of the paper. The design and the date of the drawing speak for themselves. Every viaduct built by Clarke, Reeves & Co., including the greatest of all—that at Kinzua—was built after this design, which is now being copied by European engineers. The plate is reproduced from a tracing of the original drawing, now in the Phœnix Bridge Company's office at Phœnixville.

A. P. Boller, M. Am. Soc. C. E.—Mr. Greiner's paper on the genesis of the modern railroad viaduct seems to cover all the main points of their origin and development, and leaves little more to be said on the subject. The paper is so suggestive of collateral history in the development of iron bridge building in America, that it may be well to put on record some unwritten chapters of that history made by men who were important factors in shaping the line of development in the early days of the art. Through purely accidental causes the three great nascent railway systems of those days, adopted three separate ideas of bridge building—the Baltimore and Ohio school, under the lead of Albert Fink and Wendall Bollman, developing the trussed suspension type, of which school Shaler Smith was a brilliant pupil; the Pennsylvania Railroad school, headed by Jacob H. Linville, under the inspiration of the early writings of Squire Whipple and his pupil John W. Murphy (now deceased), who had settled in Philadelphia to engage in the building of bridges (since known as the Murphy-Whipple bridge), first with A. & P. Roberts and afterwards with Stone, Quigley and Burton, noted wooden bridge builders of the day ; and the New York Central school, under the lead of George E. Gray, which adopted the riveted lattice type through the influence, if I am not mistaken, of Howard Carroll (killed during the war), one of his assistants, a brilliant young Irish engineer who had been brought up under Sir John MacNeil, and was therefore thoroughly imbued with English ideas. Mr. Hilton was also a pupil of Mr. Gray and an associate of young Carroll, and was very active in Americanizing the riveted lattice truss, and fastened for all time that type as the standard of the New York road. Outside of this road and the tributaries which it influences, the Whipple type (often erroneously called the Pratt), excepting for small spans, has been universally adopted throughout the country as the most economical and constructively the simplest, even as foreshadowed by the retiring and

modest mathematical instrument maker, who, without precedent or example, evolved the scientific basis of bridge building in America. It may be interesting to note that Mr. Whipple's first essay, in which he showed a rational method of computing bridge strains, and the manner of using material as the facilities of the shops of the day made possible, was published in 1847. The type was set up by his own hands, the cuts made by him, and my impression is the presswork was his also. The edition was a very small one, very crudely printed and illustrated. For some cause or other the author suppressed most of the edition, but fortunately a few copies became distributed and opened the eyes of the early builders to the scientific principles of their art. Some years since, while collecting data with a view to writing a history upon iron bridge building in this country, I received several communications. A very interesting one from Mr. Whipple, the *Railroad Gazette*, published in 188—. I add in this place a communication from Mr. Linville and one from Mr. Gray, the respective sponsors for the character of the development on the Pennsylvania and New York Central railroads. They tell in their own way the story of their early efforts and the difficulties with which they had to contend. I find among my papers a characteristic letter of Mr. Whipple, which ought to be put on record for its historic references. It was written shortly before his death, and it is difficult to realize the clear, firm and legible handwriting of the original, as that of a man eighty years old.

ALBANY, January 10th, 1887.

A. P. BOLLER.

Dear Sir:

In answer to yours of the 7th, I can not do better than to quote from the advertisement of my publication of 1869.

"It may be proper on this occasion to explain the somewhat anomalous —perhaps I should say incongruous, form in which this book is presented. The original publication was a pioneer effort, prompted by a hope of being able to do something toward supplying an important desideratum. It is believed that no previous attempt had been successfully made to reduce truss-bridge construction to its simplest elements, and to determine by exact calculation the forces acting upon the various parts of such structures, and to deduce thence the proper sizes and proportions of such parts, upon known and reliable principles. This was the purpose and object of the author, which was tolerably satisfactory to himself, and met as favorable a reception as could have been expected. But perfect success in all respects could hardly be hoped for in a first endeavor of the kind, and observation and reflection early suggested additions and modifications, which it seemed might make the work more valuable and useful, insomuch that its dissemination, in its then condition, was not regarded as especially desirable. Consequently, of the edition first printed, which would have been exhausted long ago had the book been easily accessible, a considerable number of copies remain still in the hands of the author, while those which have been disposed of have mostly been parted with on direct application to him, and not through the medium of the book trade. Recently, however, circumstances and inclination favoring, the author has prepared and printed with his own

hands entirely (having turned printer and xylographer for the occasion), the present addition to the matter originally published,* whence it will be seen that the history of its production is as unusual as the character of the book itself."

1st. My work was entirely original, built up on elementary principles, without reference to, or knowledge of any previous work upon the subject.

2d. I first met John W. Murphy about the year 1850 or 1851. I learned that he thought well of my book and my bridges, whence I inferred, of course, that he was a man of discrimination and ability, and as he afterwards talked up iron bridges and Whipple bridges in Pennsylvania, he was of service to the cause and to me. Iron bridges "took" in Pennsylvania rather better than in New York, and Murphy, with others, formed a partnership for building iron bridges, and purchased my patent (covering the arch truss only), in the year 1859, or thereabouts, and built a few bridges, which they were pleased to designate as Murphy-Whipple bridges, to which I made no objections, though it has perhaps been the means of disseminating false impressions. "Murphy-Whipple bridges," properly considered, simply means bridges built by Murphy upon plans and principles originated by Whipple. My relations with Mr. Murphy were most friendly, and he conceded to me all my claims to originality in the bridge question.

3d. To the best of my recollection, the first iron bridge erected for the New York Central was over the Mohawk at Schenectady, in 1858 or 1859, or thereabouts, being designed by Howard Carroll, George E. Gray being Chief Engineer of the road. Mr. Carroll complained that he was restricted to some other plan than Whipple's, and the bridge was constructed with diagonal braces and vertical ties (*a la* Howe), as I recollect the case. When and by whom the plan was changed to a web system without vertical members, I am unable to say.

Howard Carroll was a man of fine presence, 6 feet in stature, and

---

*Copy of a card attached to my model Exhibited at the Centennial exhibition.—S. W., Printer.

THE ORIGINAL:

Made in 1846, and exhibited in the

AMERICAN INSTITUTE FAIR

In 1847.

This is the first model constructed to represent the WHIPPLE TRAPEZOIDAL TRUSS BRIDGE, and is believed to illustrate the first intelligent attempt to introduce from actual calculation, correct proportions for the several parts in Bridge Trusses or Girders, according to the duty required of the parts respectively; and, to reduce to a PRACTICAL MINIMUM the material required in the construction of Bridges.

JULY 4th, 1876. SQUIRE WHIPPLE,
ALBANY, N. Y.

every inch a gentleman. He served in the war for the Union—with a colonel's commission—was in several hard engagements, and lost his life in the service. Honor to his name.

This is about all I can say in answer to your queries. I would like, however, to call your attention to an advertisement of the Rider bridge, and its endorsement by some of our most prominent engineers (to be found in the *Railroad Journal* of the year 1848, pages 775 and 776), as showing how little attention the profession at that time had given to the elementary principles of bridge construction. It seems to have been mostly a mere matter of "cut and try," and if the thing stood, all right, and no questions asked as to whether the structure contained too much material here or too little there.

Very respectfully yours, etc.,

S. Whipple.

Philadelphia, November 19th, 1886.

My dear Mr. Boller:

I send herewith a hurried statement, given below, of my most important work in iron bridge building. During my early connection with the Pennsylvania Railroad Company, the charge of maintenance of way and construction devolved on me, and much of my time was also employed in designing and constructing shops and stations. Bridge work for the entire line, from Philadelphia to Pittsburgh, was also mainly under my charge; in fact all the iron bridges were then designed by me and constructed in the Altoona shops. This subject was new to me, the demand urgent, and my force limited, consequently I was always overworked, and my health, naturally delicate, soon became permanently impaired.

When relieved from maintenance of way, I became engineer of bridges and buildings, when the designing of the bridges and buildings for the main line and dependencies was entrusted to me. In the Keystone Bridge Company the case was similar. The executive work was done by me and the designs were mainly made under my direction. There was no time to fully mature anything, no leisure for study or calm reflection. My energies were exhausted and health broken down by the incessant demand for estimates and plans; and in traveling involved in making proposals, visiting work in progress, and in settlement of questions growing out of work under construction. I gave about two hours' time to the draughtsman who prepared the design for the river span at Callowhill street, Philadelphia, and received the first premium from the city. The original design and estimate for the Steubenville Bridge was prepared in one or two days, a statement furnished showing the size and weight of the principal members, and a list of necessary tools made out, if the work was to be constructed in the Altoona shops.

I had a preference for spans constructed with inclined struts and ties, and besides numerous small trellis bridges, designed a 262-foot span for Port Perry, but the bridge was not then constructed, and later Mr. Wilson prepared the designs. I always feared, however, that trouble would arise from the wearing of the eyes and pins resulting from changes from tension to compression and *vice versa*, under moving loads. In Middletown Bridge provision was made to compensate for the wear.

Trusting I have not wearied you with details, interesting only to myself, and too many apologies for my failures, which, perchance, I might avoid if I could live my life over again,

I remain yours very truly,

J. H. Linville.

Another letter from Mr. Linville reads as follows :

The subject of bridges was not embraced in the course of study at Union College. I pursued the regular classical course, and took many optional studies from the scientific course, including analytical geometry and the calculus. When I resumed the duties of Resident Engineer of the Middle Division, Pennsylvania Railroad, I was entirely ignorant of bridge construction and the requisite computations, but was soon directed to build, in the shops of the company, the third trusses for Duncannon Bridge, from existing patterns. This led me to inquire, and I found Haupt's book on bridges, but obtained little information therefrom. I increased the strength of the third truss mainly in the suspension rods near the ends, but the patterns precluded the possibility of securing enlarged counters. Next I built Bridge 13, near Tyrone, from new patterns, also Bridges 6 and 7, west of Spruce Creek, all arched Pratt trusses. I proposed wrought-iron upper chords, but Mr. Thomson objected. This was in 1857–59. In 1860, Mr. Thomson advertised for plans for the Delaware extension bridge over the Schuylkill River, Philadelphia. I submitted a design, all riveted lattice, also a design with wrought-iron hollow posts and with flat chord links, four abreast, distributed each side of the two suspension diagonals and between ribs on the cast bases of the posts. The trusses were double intersection and posts vertical. I had no books except Haupt's, when, by considerable hard study, I discovered the formula given in the "Keystone Album," page 29. Afterwards I secured a copy of Whipple's first book and a copy of "Bow on Bracing." I had seen a plan of a Whipple bridge erected near Troy, N. Y., and had a lithograph of a Fink bridge, 200 feet span, near Monacocy. The rest was evolved from my inner consciousness. My hollow post design was adopted by Mr. Thomson over the Phœnix and numerous other designs. The lower chords were forged by John A. Wright at Freedom, with wide, rough, flat heads, and trimmed into shape at the Altoona shops. This led to investigations and experiments by Piper and myself, when the upsetting of chord heads by pressure in moulds was invented in 1861. The patent was finally sustained over the Phœnix by the Supreme Court. The hollow post patent was also confirmed to Piper and myself, but just at the time of the expiration, we discovered that both parties had been anticipated by an English patent for rolling pipes in flanged sections.

I built the Schuylkill Bridge in 1861. The specifications given by Mr. Thomson limited the live load to 2 400 pounds per lineal foot, and required the diagonals to have screw adjustments, and the floor beams to be carried on the chords. This requirement applied to all bridges built while he was president. He would not consent to eye-bolts at both ends of suspension diagonals or to the use of suspended cross-girders, and you will find this limitation affecting all the bridges built during his time on the Pennsylvania Railroad. I constructed the bridge at Rockville and the Pequea bridge, near Leaman Place, at about the same time as the Schuylkill bridge, but with cast-iron posts; otherwise the plans are similar. In 1862, I was requested by Mr. T. A. Scott to submit a plan for a 320-foot span at Steubenville. I had, in the meantime, seen Humber's work on bridges. Mr. Thomson prescribed a live load of 2 500 pounds per foot. I urged a heavier load, and after his return from England he consented to allow 3 000 pounds per foot in this bridge and in all subsequently built for the Pennsylvania Railroad. I could not obtain the hollow posts for this bridge, and there was only a limited variety of shapes then rolled, consequently I was led to use cast-iron in the posts. Mr. Thomson was very partial to cast-iron and resisted innovations. This

is the first bridge I built with leaning end posts, although I was aware of the economy in that form. Enoch Lewis objected to inclined posts when proposed previously for the Rockville bridge. He did not like their appearance. In the Steubenville deck spans I employed auxiliary rolled beams to carry the floor beams, and this plan was adopted for bridges above Lewistown, Manayunk, Mount Union and Coatesville, the compression being taken by a cast-iron upper chord. In 1863 I advocated bridges entirely of wrought-iron in a circular issued by the Keystone Bridge Company. In 1865 I induced Mr. Thomson to adopt wrought-iron for the 262-foot span for the connecting railway bridge over the Schuylkill River, but he still adhered to cast-iron for small bridges. This was the final triumph for wrought-iron, although after this bridge was finished he ordered me to continue the counters to the ends for appearance, and to put on an arch. I succeeded, after considerable persuasion, in obtaining permission to attach the arches so that they could not support any weight. Then followed bridges at Easton, for the Lehigh and Susquehanna Railroad Company, and the Newport and Cincinnati bridge, of which I was chief engineer. We were making octagon hollow posts and upset chord links in 1865. The Newport and Cincinnati bridge had a span of 420 feet, to carry 4 500 pounds per foot, and a pivot span of 360 feet, afterwards abandoned when the Government required us to elevate the bridge from 70 feet (as designed and partly erected) to 100 feet above low water. This ruined the general design and necessitated patch-work expedients to save the work, then mainly constructed and partly erected.

After this followed the channel spans of the Benwood and Parkersburg bridge over the Ohio, and of the Dubuque and Keokuk bridge over the Mississippi. The Keokuk pivot span was 378 feet. I do not mention the multitude of short span bridges mainly designed by me for the Keystone Bridge Company. The next notable structure was the Schuylkill River span at Callowhill street, for which was awarded to me the first premium from the city. This bridge was ruined in appearance by the requirement of cast-iron arches (afterwards changed to galvanized iron) to match the design made by Mr. Kneass for the approaches. The Raritan Bay pivot span, 472 feet in length, was designed about this time. Then followed the channel span, 520 feet, for the Cincinnati Southern Railroad Company over the Ohio River at Cincinnati. I condemned Captain Eads' first design for the St. Louis bridge (which ended my connection therewith as consulting engineer), and suggested arches in depth one-fortieth of the span, with the material in tubular form, instead of his solid rib about 8 inches square.

Aside from my connection with elevated railway work and the Kinzua viaduct, I do not recall anything further worthy of mention. I always favored heavier bridges and numerous changes in details, which I was prevented from making, owing to the close competition and the requirements of manufacture. Our "Keystone" practical men were averse to change, but gradually fell into the use of wrought-iron box posts and chords, and later, latticed posts. The subject became tiresome and monotonous to me as I became worn in health and spirit by overwork, and I do not believe anything short of a phenomenal structure could again arouse my interest in bridge work. If you will refer to Clarke's book on the Quincy bridge and probably Merrill's book and many others, you will find strains allowed in counters at the extreme ends of spans. I do not claim this as a brilliant mathematical achievement, but considering that it was original, and developed by laying awake one night and without any aid from books, and at a time when I was in a deplor-

able state of ignorance, I always felt somewhat proud of the result. Three years ago I made a bonfire of numerous letter-books and bridge plans, as they were in my way in the country. My Keystone letter-books were shipped to the Keystone Bridge Company, Pittsburgh, and are in the attic. A fire destroyed my house and library some years ago, and I can only give you this hurried sketch from memory. I care nothing for fame, and only fear that your generosity and friendly feeling may lead you to give me greater credit and praise than I merit, as I utterly despise all that I ever constructed, and with the present light cannot comprehend why I did not accomplish something better. I presume every tyro from the schools now wonders why mistakes, now so apparent, were made, but no one can realize and understand these things who has not groped in untrodden paths as did the early builders of American bridges.

MAY 23d, 1887.

Memoranda relating to the introduction of wrought-iron bridges (lattice) on the New York Central Railroad, made by the late Chief Engineer, George E. Gray:

The engineer of this period cannot fully realize the embarrassment his predecessors labored under to keep pace with the march of improvement of the country by the introduction of railroads as a means of transportation. In no particular case was this more evident than in the designing and building of bridges and other structures of like character on the New York Central Railroad, prior to 1853, the year the several companies between Albany, Troy, Buffalo, Niagara Falls, etc., were consolidated. In that year the writer was appointed Chief Engineer of the consolidated company, the New York Central. His powers and duties were limited to the designing and constructing new works, and when completed, turning the new work over to the local superintendent's care and maintenance.

This condition of responsibility continued until 1858. On May 11th, 1858, a serious accident occurred at the Saquoit Creek bridge, about four miles west of the City of Utica, resulting in great loss of life. This accident brought to the attention of the directors of the company the subject of where the responsibility rested for the care and maintenance of the bridges on the line of the road. Investigation brought out the fact that there was no organized system of inspection of the bridges. The Chief Engineer was then requested to suggest a method of inspection, which was approved by the Board of Directors by resolutions requiring the Chief Engineer to make a careful inspection and test once in six months, and report to the directors such recommendations as seemed proper. In addition, the local superintendents and roadmasters were to make independent inspections every three months on their several divisions and report to the Chief Engineer. Under this system the first inspection brought to light a most lamentable state of the condition of the bridges. The several companies forming the consolidated company had constructed their roads on very economical methods. The bridges were of every conceivable design, and when the designer had knowledge and experience to proportion the parts properly, the load provided for contemplated engines not exceeding 20 to 25 tons, the latter considered heavy. Then, bridges had been neglected, many vital members were badly decayed, important bolts and tie rods were rusted and not properly adjusted, and as a whole, were unequal to carry safely the fast increasing traffic. This condition as found, the perishable character of the material

(wood), and the frequent renewals (unless covered in, increasing risk from fire, etc.), led up to the question, Is there a more durable material for bridges? Can we substitute iron and stone within a reasonable cost? Iron bridges were the order of the day in Europe, but their reputed cost disheartened our directors. The income would not warrant the outlay. Again, the lack of proper information prevented a full understanding of the subject. At that period there was not an all wrought-iron bridge on any railroad within the United States (to the knowledge of the writer), except an example or two of short-span plate girders. My old friend and mentor, Squire Whipple, of Albany, N. Y., had designed and constructed a few combined cast and wrought iron bridges. The directors of the New York Central Company had great distrust of iron for bridges, but in particular of cast-iron, and they would not entertain its use. This led to the study of wrought-iron suited to the purpose of the New York Central. The only hope was to so reduce cost as to come within the means of the company, and this could be done only by reducing quantity of material.

The European engineer had abundance of capital and cheap labor; the American engineer had the cost of the article, to which must be added transportation, custom duties, and the high price of labor, both skilled and common; and another difficulty, a want of information and experience in iron bridge building. There were no civil engineering societies to bring civil engineers together where they could exchange ideas and tell their experiences. The literature on this subject was limited. The writer sought every known publication at home and in Europe, and when all was sifted it was not satisfactory beyond demonstrating that wrought-iron, as used by European engineers, was beyond the financial power of the New York Central Company. However, the query came to the writer's mind, why not cut out what seems unnecessary material in European plans?

Working from this idea, and from data obtained, plans and calculations were prepared for a few short span lattice bridges. The best forms for compression and tension members and for the joining of members were designed. The formulas for calculating compression strains, without bending, etc., were wanting or unsatisfactory. Fairbairn and others gave compression formulas, but nothing satisfactory for deflection. A halt was made until a testing machine could be made, and then the several sections designed were tested, and from these tests formulas were obtained from which all the designs for lattice bridges on the New York Central were developed. When designs had been prepared it was discovered that very few of the forms or sections desired could be had, except at extra cost for the rolls; rolling mills did not wish to undertake such small invoices, and only limited sizes of flat bar and angle and T iron could be had from merchant stock. Many makeshifts had to be resorted to in filling a bill of iron for any bridge, and many changes involving new calculations became necessary. The engineer who may now be called upon from curiosity or investigation to examine those early examples must in justice bear in mind that nearly all the sections or forms he may see are not the original choice of the designer, but forms forced upon him from the want of better. The increasing traffic admonished the engineer to meet the demands for heavier rolling stock, etc. Therefore the new designs prepared contemplated 1½ tons per lineal foot of single track. The designing and construction proceeded cautiously, but steadily. Small spans were first constructed (and these attracted the attention of railroad officials), notably the bridge over the Mohawk River at Schenectedy and the Hamburg Street Canal in Buffalo. These bridges met our expec-

tations; superintendents and roadmasters approved them, and asked for more of the same sort. The Board of Directors began to feel confidence in wrought-iron, and indorsed by resolution the recommendation of the Chief Engineer to replace all wooden bridges by iron as fast as they required renewal. About this time rolling mill managers began to see the growing importance of iron in bridge building.

Among the first long span bridges erected was a double track bridge, of 125 feet span, over the Tonawanda Creek at Batavia, N. Y. This bridge was constructed and brought into use in 1860, the plans having been ready a year before. This structure has but two trusses, the floor beams being of depth to carry the weight of both tracks. The location is near the home of the late Dean Richmond, Esq., the then Vice-President of the New York Central Company. Mr. Richmond watched the progress of this bridge with deep interest. When completed and tested by the heaviest engines then in use, filling the two tracks the entire length and speeding the engines, all of which it bore to our entire satisfaction—Mr. Richmond being present, he turned to the writer and said: "Well, Mr. Chief Engineer, there is a monument for you that will last long after you and I have gone to dust. That is the kind of bridge I am in favor of; no more wooden bridges for this company."

After the completion of the Batavia bridge others followed rapidly, confidence seemingly fully established, the main conflict being the great cost. The next in order was a double track bridge on the Erie Canal near Newark, another on the Erie Canal at Rome, and another on the Erie Canal at Schenectady. The first iron bridge was erected in 1859 and 1860. During the fiscal year ending September 30th, 1862, there were constructed 697 linear feet, making a total to that date of 4 $670\frac{1}{6}$ linear feet equivalent, single track. September 30th, 1863, 499 feet were added, making a total of 5 $169\frac{1}{6}$ feet; September 30th, 1864, only $130\frac{6}{12}$ linear feet were added. During the year 1865, the total linear feet of equivalent single track on the whole line was 5 $677\frac{8}{12}$ feet. In addition to the long spans constructed, a large number of small spans were built, forming in the aggregate a formidable bill of expenses chargeable to bridge renewals.

The duties of his position as Chief Engineer of the New York Central Railroad (always arduous) required additional assistance other than the ordinary routine. Mr. Howard Carrol, a young Irish civil engineer, who had served under Major McNeil in Ireland, was employed. Mr. Carrol had some experience in the erection of small span plate girders. His experience, fitness by education, and his inclinations made him a valuable assistant. From the data already obtained and the writer's constant personal supervision, the first design and the calculation of strains and tests for the lattice bridges were brought forth, and the bridges constructed as now seen on the New York Central Railroad. Mr. Carrol continued in this special duty until his resignation. Mr. Carrol's love for his adopted country prompted him to assist in raising a regiment of New York State volunteers for service in the war of the rebellion. With this regiment he went to the front as the Lieutenant-Colonel. It was in this serving, while gallantly leading his brigade at the battle of Antietam, that he received wounds from which he died a few days after in Washington, D. C. Thus died a noble man and a ripe scholar, who, had he lived, would have made more than an ordinary mark in the history of iron bridge building.

Mr. Chas. Hilton was another assistant serving with Mr. Carrol, and succeeded the latter after his resignation. Mr. Hilton succeeded the writer when he resigned his position of Chief Engineer, to take that

of Constructing Engineer to the Central Pacific Railroad of California. He continued the designing and construction of iron bridges on the New York Central until his retirement, and afterward devoted himself to constructing and building iron bridges, making some modifications in minor details.

J. E. Greiner, M. Am. Soc. C. E.—The author regrets that the original intention of entering into the details of viaduct construction was not carried out, because there is sufficient evidence in the foregoing discussion to indicate that such a diversion would have been pretty thoroughly ventilated.

In answer to Mr. Moore, the author did not intend to convey the impression that the single leg type of structure is cheaper than any other in all cases, but can see no reason why Mr. Bouscaren's opinion that, "circumstances permitting its use, it is superior to others," should not be correct. The fact that a particular case figured by Mr. Moore required an excessive amount of material proves nothing to the contrary.

Concerning the proper lay-out of spans, the engine loads, material used and cost of the same erected, enter into consideration, and the economical lengths for light engines will not always be economical for heavier loads. For instance, a single track viaduct proportioned for the 107.5 ton Baltimore and Ohio consolidated engines will weigh very nearly, for 30 x 30 feet spans, 440 pounds per linear foot, and 340 pounds per vertical foot at 30 feet intervals; the height being taken under the girders. For 30 and 60 feet spans the weight would be 650 pounds per linear foot, and 400 pounds per vertical foot of bents. The height of the legs which will give the same weight of iron for each case can be approximately determined from the formula—

$$2.4\ h\ s + 60\ h = 210\ s$$

where s = the total length of structure, h = the average vertical height of legs to give the same weight of structure.

Thus, when

$s = 360$ feet.	$h = 81.7$ feet.	$s = 720$ feet.	$h = 84.6$ feet.
$s = 540$ "	$h = 83.6$ "	$s = 900$ "	$h = 85.1$ "

If it is assumed that there will be about 7 cubic yards of masonry to each bent, that masonry costs $12 per cubic yard, and iron 4½ cents per pound erected, then the cost of the two cases will be the same when $102\ h\ s + 2.5\ h = 8\ s$ gives for

180 feet spans.	$h = 69.00$ feet.
360 " "	$h = 73.44$ "
900 " "	$h = 76.4$ "

From the above it appears that the length of structure, as well as the height, will influence the lay-out.

There are many interesting points in viaduct construction which might be discussed, but this is not the time.

Mr. Moore seems disposed to criticise the author's limitation of the term viaduct to a structure of metal or wood supported by braced

towers. Of course such a limitation would exclude the stone viaduct, still there would be no objection to the classification of such bridges as "stone arches." The etymological meaning is too general and is not the one usually accepted. In this sense "a viaduct is a structure carrying a road." Then, according to this, the East River Suspension and the Niagara Cantilever bridges, as well as the Kinzua, are viaducts. No one, however, refers to the suspension or cantilever bridges as viaducts, and this clearly proves that the accepted meaning is limited to certain kinds of structures which cross ravines where there is little or no water. The definition given by the author is, perhaps, too narrow, and was advanced merely to bring out the views of others.

The collateral history on the development of iron bridge building in America, furnished by Mr. Boller, is by far more interesting than the author's attempt. In reading of these early efforts made to correctly design bridges, one necessarily feels a veneration for those great pioneer bridge engineers. Perhaps, as Mr. Linville says in his letter, "some tyro fresh from school wonders why it was possible to make mistakes now so apparent," but the same tyro invariably outgrows his wonderment. Every engineer worthy the name fully respects such men as Whipple and Linville.

In conclusion there is appended hereto a letter from Mr. Bouscaren, relative to the designs of the early viaducts on the Louisville and Nashville road, which explains itself :

Dear Sir :

Referring to your remarks on the viaducts of the Cincinnati Southern Railway, I desire to state that the idea of substituting alternate 60 and 30 feet spans in place of all 30 feet spans, was first suggested to me by Mr. C. L. Strobel, who was at that time (1878) my assistant on the Cincinnati Southern Railway. He should receive due credit for the idea.

The Horse Run Viaduct mentioned in your paper was designed and built by the Keystone Bridge Co., under the condition to preserve the general feature as to longitudinal bracing, shown in your diagram.

Yours very truly,

G. Bouscaren.

Cincinnati, July 16th, 1891.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

TRANSACTIONS.

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Vol. XXV.—October, 1891.

SCREW STEAMSHIP AND TOW BARGE EFFICIENCY ON THE NORTHWESTERN LAKES OF AMERICA.

By JOSEPH R. OLDHAM, N. A., Cleveland, O., 1891.

WITH DISCUSSION.

The subject of the "American Whaleback Steamers" and towage on the northwestern lakes is one of great interest at the present time. About twenty-five years ago there were built in London several vessels having circular midship sections, and extremities somewhat resembling these "whalebacks," but it would seem that their demise followed rapidly upon their inception. These vessels were called "cigar ships." Again about midway between that period and the present date another cylindrical floating iron vessel was constructed to carry the Egyptian Obelisk, or Cleopatra's Needle, to London. This vessel, after an adventurous voyage over the Bay of Biscay, appears to have accomplished her life's work in one single effort of usefulness. But not to such as these may we liken Captain McDougall's invention, for within two years of regular building not fewer than 70 000 tons of shipping is represented by these ubiquitous barges.

It was said some time since that the northwestern lakes would

soon be flooded with the McDougall barges, and now it really seems as if this prophecy would very soon be consummated, for in his last trip up these lakes the writer passed no fewer than three tows of these vessels, besides the *Colgate Hoyt*, and the tow with which he came quickly and pleasantly down the lakes. About this tow something more will be said later on. In the meantime it may be stated that this flooding of the lakes with whalebacks is not looked on complacently by either our ship-builders or ship-owners. As the question is often asked, the author desires to state his opinion that, given sufficient free board, strength and power, the North Atlantic trade may be safely carried on in such vessels.

Good speed should be a *sine qua non* when equipping a properly loaded steamer for running before an Atlantic gale. Form taken by itself is but a poor condition of safety, for a good model will not behave well in a seaway if overloaded; and an inferior model may do fairly well if she retains sufficient surplus buoyancy. This remark should be qualified by saying that ordinary cargo steamers only are now referred to.

There is nothing more suprising about these whaleback steamers than their extraordinary speed, considering the low power and small consumption of fuel required, unless it is the ease with which the whaleback barges tow, when we take into account their large capacity and enormous dead-weight ability. For example, the steamship *Joseph L. Colby*, with only 850 indicated horse-power, can tow not fewer than three of these vessels nearly 8 miles per hour. This would not be bad work if such a boat were light, but when we learn that this can be accomplished with 400 000 bushels or nearly 9 000 tons of wheat on board, the efficiency may surely be called phenomenal; for though it is generally assumed that the thrust from a screw propeller is greater than the tow-rope pull, this is not always the case. It is frequently stated that the thrust of a propeller does not exceed 60 per cent. of the horse-power developed, but in the Admiralty experiments with the screw steamer *Rattler* the efficiency was as high as 80 per cent. Some particulars of this steamer and her trials may here be quoted. The indicated horse-power of her engines was 428. The thrust of the shaft, with a dynamometer which should measure the forward effort of the screw was about 8 000 pounds. If this be multiplied by 1 013, the speed of the vessel in feet per minute, and divided by 33 000, it gives 245 as the horse-power utilized; and assuming that the indicated horse-power was 428,

this is only a little over 57 per cent. of the indicated engine power. It is said that the efficiency in this instance fell on account of a strong head wind, though the thrust was then greater than when the efficiency percentage was 84.

Now, if we assume an equal state of efficiency in the screw steamer *Joseph L. Colby*, it will still be somewhat phenomenal to see her steam off with three of these loaded barges as she does. But let us take an example of these barges being towed by an ordinary lake steamer. A few weeks since it was the author's good fortune to make a trip from Lake Superior in the handsome wooden steamer *Sitka*, of the Wilson Line. The barges 105 and 109 were in tow, with a combined cargo of over 4 700 tons dead-weight on board. The *Sitka* carried in addition 1 900 tons, including her fuel; hence, the total dead-weight carried was 6 600 tons, and the gross displacement of all three vessels would not be less than 10 600 tons, or about the same as the ocean mail steamers *City of Rome* or *Servia*. A careful timing of the vessel proved beyond a shadow of doubt that the *Sitka* steamed with its tow fully $8\frac{1}{3}$ miles per hour on the average. But let us take some other means of comparison before we lay claim to this high efficiency. The registered dimensions of the *Sitka* are 272 x 40 x 19 feet, and her tonnage is of the gross register 1 741 tons. She has triple expansion engines, the diameters of cylinders being 20, 32 and 52 inches respectively, and common stroke of piston 40 inches. The boilers are of the Scotch type and are 132 inches in diameter by 12 feet in length, and they have four flues or furnaces, each 46 inches diameter, the working pressure being 150 pounds per square inch. The author was not able to ascertain the mean indicated horse-power exerted by the engines during the run measured, and cannot, therefore, say what was their minimum power, but can estimate her maximum power without inaccuracy. Lloyds give the following rule to be used for determining the nominal horse-power of triple expansion engines.

Rule: $\frac{1}{2}\left(\frac{D^2 \times \sqrt{S}}{100}\right) + \frac{H}{15}$ = nominal horse-power of triple expansion engines. Where D = the diameter of the low pressure cylinder in inches, S = stroke of piston in inches, H = the heating surface in square feet. If the result be multiplied by 5 it will show a power in excess of good practice for similar engines and boilers in ordinary working. By this rule, allowing the heating surface of the *Sitka's* boilers to be 3 000 square feet, the engine would be developing $927\frac{1}{2}$ horse-power. However, as

the author is not certain as to the exact effective heating surface, let us test this by a rule of our own, which is :

$$\tfrac{1}{5}\left(\frac{D^2 \times \sqrt{S}}{10}\right) \times (F^2 \times N \times 50) = \text{I. H. P.}$$

Where D and S are as before, F = diameter of furnace in feet, N = number of furnaces. By this formula the power would be 929.6 horse-power.

Permit the quotation of one more rule for approximating the horse-power of marine engines, and then we may not hesitate to give the limit of the power of these engines, or indeed any common marine screw engine in ordinary working without mechanically forced draft:

$$\text{I. H. P.} = \left(\frac{D^2 \sqrt[3]{S} + 3H}{100}\right)\sqrt[3]{P}.$$

Where D = the diameter of the low pressure cylinder in inches, and if there is more than one low pressure cylinder, let D = the sum of the squares of the diameters, S and H are as before, P = the working boiler pressure in pounds per square inch above the atmosphere. By this rule these engines would indicate 969 horse-power, and the mean power by the three formulas would be 942. Let us be liberal, however, and assume that these engines were indicating as much as 950 horse-power. In connection with this it must be mentioned that the height of this steamer's stack would be nearly 60 feet ; and if we assume that every 10 feet in height supplies a draft equal to $\frac{1}{16}$ inch of water pressure, there would be about $\frac{3}{8}$ of an inch total pressure. The furnaces were large in diameter, there being wisely but two fires in each boiler. Parenthetically it may be remarked that the man who attempts to work three corrugated furnaces into a boiler not more than 12 feet in diameter, would become a better designer if he spent some time as a "shovel engineer." For, remember, that the outside of the corrugation limits the cleaning and water circulating space, whilst the diameter inside of such corrugations limits the diameter for fire bars as well as the opening for the passage of air; and if we allow, say, 10 inches depth of coal on top of such fire bars, we shall not then wonder at a low efficiency as regards consumption of fuel, if we bear in mind that the first duty of a furnace is to burn coal. It will be observed that the diameter of a furnace, or breadth of fire, only, enters into this calculation; for it does not appear that the length of furnace makes any appreciable difference in the power of a boiler; indeed, it is not an uncommon experience with marine

engineers to find the length of furnaces decreased with a corresponding increase in consumption of coal and power developed. Again, who has not known (at least with boilers that have been overtubed) of an increase of power following upon a reduction of heating surface? As to the power of the engine, we have taken the low pressure engine only in estimating this, for that only is necessary.

The indicated horse-power required to attain a certain speed may be ascertained from the following expression.

$$\text{I. H. P.} = \frac{\text{Speed}^3 \times \sqrt[3]{\text{displacement}^2}}{\text{co-efficient}}$$

This is based on the rule for the co-efficient of performance of a steamship in relation to the displacement, which says the cube of the speed in knots per hour, multiplied by the cube root of the square of the displacement in tons of the ship and divided by the indicated horse-power, equals the co-efficient of performance.

In the last column of the annexed table will be found constants applicable to the above formula for various types of screw steamers, having speeds from about 8 knots to nearly 25 knots per hour. Now the *Sitka*'s average speed over the voyage under consideration was 8½ miles per hour; but thick weather and head winds lowered the average, for when such did not prevail the *Sitka* and tow could steam nearly 9¼ miles per hour, or slightly over 8 knots; hence, to equal this, our proposed vessel should be able to steam 8 knots per hour. Let us estimate what horse-power will be required to attain this speed approximately by another formula, based upon skin frictional or augmented surface resistance:

$$\text{I. H. P.} = \frac{S^3 \times L \times B + 2\,D \times 0.91}{20\,000}$$

where S = the speed of the vessel in knots per hour, L = the length and B = the breath of the vessel, D = the draft of water (mean). The divisor 20 000 expresses the number of square feet of augmented surface which can be driven at a velocity of 1 knot per hour by one indicated horse-power.

According to this rule the power required to transport 6 600 gross tons dead-weight of cargo on one bottom, at as great an average speed as that which the *Sitka* and her tow of two loaded whalebacks accomplished, would be 1 189 horse-power. By this it appears that an equal amount of freight can thus be carried in these three hulls with 20

per cent. less horse-power; and 20 per cent. saving in a steamer like the *Sitka* equals nearly 5 tons of coal every twenty-four hours.

Now, if we had to carry an equal dead-weight in one vessel on the same draft of water, a steamer 580 feet in length and 58 feet in breadth would be required. To drive this steamer at 8 knots per hour, engines of 1 189 horse-power would be necessary, which is certainly 20 per cent. more than the power of the *Sitka*, which propelled and towed 6 600 tons of dead-weight of cargo at an average speed of more than 8½ miles per hour. To compare this vessel with the whalebacks as cargo-carrying machines, we must content ourselves in this instance with the gross displacement in each case, indicated horse-power, and speed of vessel; and this may best be done by the aid of the well-known formula above referred to, $\frac{V^3 \times D^{\frac{2}{3}}}{\text{I. H. P.}}$. This formula is based on the assumption that the resistance offered to the passage of a vessel through water is proportioned to the square of the speed; that the power required to propel the vessel varies as the cube of the speed, and as the cube root of the displacement squared. Although this may not be absolutely correct, it is quite near enough for our purpose at present, and as the resistance experienced by the fore end of a vessel varies exactly as the square of the speed, there is no reason for supposing that in properly formed vessels the after body should materially interfere with this method of comparison. By this formula we find that the efficiency of the equivalent steamer is represented by 213, and that the efficiency of performance of the whaleback tow is 247, which is much higher.

These vessels transported on this occasion one gross ton a distance of 100 miles for a consumption of 3.65 pounds of coal at 8½ miles per hour; at a 10-knot speed, the best average the writer is acquainted with, equals one gross ton carried 100 miles for every 5 pounds of coal burnt. Of course this great discrepancy is largely attributable to the higher speed, the power to attain which varies as the cube of the velocity; but even when discounted in this way the result is still worth noting, for it represents in cost of coal one ton of freight carried 100 miles for less than one cent. Incidentally it raises the well-beaten problem of the most economical speed for a freight steamer. The author considers the average speed of modern lake steamers to be too high, as they are not required to run before an Atlantic gale.

The great length of tow-rope used on the lakes (frequently as long as

120 fathoms) has somewhat surprised the author, for he thought the nearer the power approached the resistance the greater the efficiency. We may assume, however, that these people with great experience know what is best, but there is probably another good reason for working with a long tow-rope. Rankine says, "that propeller is the best which drives astern the largest quantity of water at the least velocity;" or again, "the useful work of the propeller is proportional to the backward acceleration of the wake." Other authorities maintain that such water is left astern. We are assuming the vessel to be moving ahead, of course, not astern, but if it be true that the pressure on the after surface of the screw-blades is attained by the production of a partial vacuum on the fore side, it would seem that the water must be driven astern, perhaps not quite in the form of a regular twisted rope, as we often see illustrated, though that the screw-blades do give the water in contact with them some rotary motion appears from observation and deduction quite incontrovertible. There is no denying that the bow of a floating vessel, when in motion, does impart like motion to the film of water of more or less thickness which impinges on her skin. If the hull can thus impart motion to the water in the direction of its advance by the medium of skin friction, does it not follow that the screw-blades may also impart rotary motion to the film of water directly impinging on their surface as they revolve? Thus it would seem to be advisable to allow the "tow" to remain as far away from the water put into motion by the screw as possible, as well as not to interfere with the wave of replacement as it advances on to the stern of the propelling vessel.

With reference to the cost of towage the author may mention that the rule on these lakes is for the steamer to receive one-third of the freight due to the barge, but the McDougall barges are all being towed for one-fourth of their freight.

Self-Trimmers.—The author calls these vessels self-trimmers, which name was given to certain steamers on the Tyne; and for this reason wing boards were fitted from the top sides, extending up to the deck and diagonally towards the center line of the vessels. Their province was to take away the vacant corner that is always left in an ordinary vessel, just at the junction of the top with the sides and deck; but the designer does not believe in vacant spaces at all, and so instead of hiding this corner, as it were, he takes the bold step of cutting the corner off altogether, thus enabling the cargo to fill the hold as full as

an egg, and that without padding of any kind. Thus these vessels may truly be called self-trimmers.

As regards collisions, if the object, when designing this bow, had been to produce the form of least destructive power, it could not have been accomplished better, for this is just the opposite of the ordinary upright stem or "ram" bow which so frequently does greater damage below than above the water line.

Seaworthiness.—It may be remembered that when the official inquiry was held over the foundering of the then new mail steamer *London*, which went down with passengers and crew in the Bay of Biscay, it was proved that the foundering was caused by a spar breaking loose from its lashings on deck and knocking against the engine-room skylight till it gave way; the water found its way into the engine and boiler compartments, put out the fires, and the vessel became a helpless mass in the trough of the sea. From this it would seem that even the great spread of canvas carried by the *London* was practically useless to her in time of need. But what the author desires to emphasize is, the extreme danger there is in having light upper works, especially in "well" deck vessels.

At the twenty-seventh session of Naval Architects, Professor Elgar in his most interesting and instructive paper entitled "Notes upon Losses at Sea" described the foundering of a certain steamer, the principal dimensions of which were 260 feet 2 inches x 34 x 23 feet 9 inches and having 4 feet 9 inches freeboard when loaded with maize (which is a cargo of approximately the same density as wheat). A heavy sea was shipped, which smashed in deck-houses and sky-lights and found its way into the engine room and stokehold; other seas followed, tearing ventilators off and pouring into the engine-room and holds. She broached to and fell over to port, her lee rail being under water. The pumps were choked and the engines ceased to work, and the vessel was finally abandoned. The above description of foundering would no doubt be applicable to many a missing steamer if we knew the particulars leading up to their loss, hence it would seem that the most vulnerable parts of such steamers are their skylights, companions and deck-houses. The "whalebacks" are almost free of such openings to the holds, those that there are being raised far above the deck on the most substantial iron turrets or trunkways. It has been said that in all things, but proverbially in mechanics, simplicity is

supreme excellence. Now, surely, we have excellence here, for nothing could be more simple (consistent with strength and handiness) than these very vessels; there is absolutely nothing to be washed away. It may be said you have nothing to keep the sea off; well, if one has to walk over a rough road when it is raining in torrents and blowing a hurricane, one would much prefer to have a good macintosh coat on his back than to trust to the frail protection of a flimsy umbrella, and so a good strong deck, without weak incumbrances, is a much safer arrangement for a storm than all the usual paraphernalia of wood bulwarks and skylights as they are commonly fitted. Big seas will get on board, do what we may to prevent them, and it is the big seas that must be provided against, not the small ones. When such do come on board the less obstruction there is to their movement in the way of side houses, etc., the better. More than that, when the seas come up the sides of vessels like these they are water borne and do not sink the hull down ready for destruction by a bigger sea; while in the ordinary vessel, with bulwarks, etc., the seas that do the damage are those that are cut off from their fellows and make a lodgment on deck by filling the "wells," when, if they do not carry all before them, they sink the vessel down, ready for the reception of another wave that does the damage. Instead of theorizing let us take an illustration of the superior seagoing qualities of these barges. During one of the most severe gales we have had on the waters of the lakes, two of these whalebacks were on Lake Superior, making good weather, considering all things; for so fierce was the gale and heavy the sea, that a new schooner, in good trim, foundered in sight of the whalebacks, while the latter are here now, having weathered the storm without damage or danger. As regards our steel lake steamers, they are quite fit to encounter with safety an Atlantic storm as they are ordinarily loaded on these lakes, but if they ever come to be loaded down, like "three-deck" tramp steamers, it will, I believe, be found that their deck-houses, companions, skylights and hatches are quite unequal to withstand the seas of the North Atlantic. No fewer than 41 British steamers were lost during the one disastrous month of December, 1872, the most serious losses occurring to low powered grain laden vessels in the North Atlantic. The *Teutonic*, with a bow 32 feet out of water, is none too good to face a severe storm in that ocean.

Now, a word about the construction of these peculiar vessels. The

author, from a careful examination, would say that he has never seen better work. He would particularly notice the ballast tanks, which are most carefully planned and constructed thoughout and amongst the best he has seen in America. This is saying a good deal, for the most important feature in a steamer next to the bottom itself is the double bottom arrangement.

As regards their stanchness he would relate some facts which appear very creditable. Four of them each loaded cargoes of wheat on their first voyage and delivered the whole freight, after carrying it over 1 000 miles of stormy water, without the slightest damage. It is an uncommon thing to find any steel vessel perfectly water-tight on her first voyage, but here we have a fleet of four vessels which did not leak a drop from the beginning to the end of their first voyage. The designer may be justly proud of this achievement.

The author has long maintained that to an engineer, at least, that what is satisfactory looks well, or should do so. So when any one says, for instance: "Oh! I would not have a bow like that, it does not look nice," the reply is, "If that is all your objection it is not worth serious consideration." But let us see if it is nice. Ask any good naval architect this question as to their appearance. Look at that bow. Can you improve it by adding the ordinary dead-wood? Would such not render the vessel less handy to steer and add useless weight to her bow?

Let us examine the question of their rolling qualities. The fault with our broad lake steamers is that they have too much stability of form. This is brought about by the center of buoyancy moving out from the center too rapidly and too much as the vessel heels over. The best way of counteracting this would be to cut off the upper corners of the gunwale, or rather to curve them in just as the American steel barges are formed. This would cause them to roll much easier, or, in other words, make them better sea boats. As regards stability, it may be said that so long as the present maximum ratio of breadth to depth is maintained, there would seem to be little or no cause for anxiety about their stability even with a full cargo of grain, if fairly loaded. This is not a mere statement nor yet a deduction, for the *Charles W. Wetmore* has recently steamed across the Atlantic Ocean in ballast, without any cargo in the hold, and with over 600 tons weight of coals on her 'tween-deck beams, which are situated 16 feet above the keel.

In conclusion, the following may be cited as amongst the desirable

improvements in lake steamship construction, as being within the range of practical achievement during the next few years in this center of the world's progress in mechanical science.

First.—Greater length and breadth are necessities in steamers.

Second.—Greater simplicity in design and construction are desirable.

Third.—Deeper water ballast tanks, and bottoms to be more carefully arranged to withstand vertical longitudinal stress.

Fourth.—Top sides and upper works to be designed more particularly to resist increased stresses due to greater length in proportion to depth.

Fifth.—Hold bulkheads to be made stronger generally, and stiffened as may be required to resist the hydrostatic pressure. The bulkhead to be more efficiently connected to the shell of the vessel than generally obtains; also an improved system of bulkheading is advisable, with a view to avert sudden disaster after violent collision.

Sixth.—A more efficient system of bilge and ballast pump, suction pipes and valves is desirable.

Seventh.—Uniformity in handling or turning steering wheels is a necessity if collisions are not to increase.

Stockless anchors to stow in the hause-pipes would save labor. Shorter stroke in engines and fewer tubes in boilers would give increased efficiency.

Improved rules for the construction of steel vessels and a proper system of inspection are necessities if marine insurance is to continue a possibility with the insured and the insurer, for the ship-owner pays for the losses in the long run, the underwriter's business being a commission on the amount of loss paid; and the greater the amount of loss in the long run, the greater would be his commission on the loss.

The following statement is made by J. McArthur, Master of the *Colgate Hoyt:*

We arrived here last night and will leave here to-night. We were just eight days in making the round trip with consort, and we laid twelve hours at two harbors. We find in running alone that our average fueling, for three trips, was 122 tons per trip. In towing consort we made our trip with 140 tons. I think this is very economical on fuel. We tow about 11½ miles per hour light, and 9¼ miles loaded.

NOTE.—Plate LXXXIV and LXXXV are the only ones showing details which the author has been able to obtain. Plate LXXXIII gives a good general view of one of the boats.

GENERAL PARTICULARS, SPEED AND EFFICIENCY CONSTANTS OF SEVERAL TYPICAL SCREW STEAMERS.

NAME.	Length.	Breadth.	Depth.	Draft.	Tonnage V. D.	Displacement.	Horse-power.	Speed per hour.	Constants.
	Feet.	Feet.	Feet.	Feet.		Tons.	I. H. P.	Knots.	$\frac{D^{\frac{2}{3}} \times S^3}{\text{I. H. P.}}$
Teutonic*.....M	565.8	57.8	39.2	26.0	8 829	13 000	17 000	20.18	267.3
City of Rome.....M	560.2	52.3	37.0	21.4	7 468	11 230	11 890	18.235	255
City of Paris.....M	527.0	63.0	39.2	.....	8 570	12 500	20 100	21.952	283
Servia.....M	515.0	52.1	37.0	23.3	7 212	10 960	10 300	17.00	234.3
Etruria.....M	501.5	57.2	38.2	22.5	7 129	9 860	14 320	20.18	260
Oregon.....M	501.0	54.3	38.0	23.7	7 017	11 000	13 300	18.30	227.9
Columbia.....M	463.5	55.6	35.5	.....	.....	10 106	12 850	19.77	277
Britannic.....M	455.0	45.2	33.7	23.0	4 757	7 000	5 400	16.00	277
Tyne.....G	319.0	34.0	26.0	18.0	.....	3 445	1 194	11.59	297
Jubilee.....C	300.0	40.0	27.5	23.8	2 230	5 800	1 360	10.00	237
Mariska.....L C	300.0	40.0	24.5	14.0	.....	4 100	1 234	10.9	268
Brunswick.....C	246.0	33.0	18.0	18.0	1 132	3 000	650	9.0	234
Bathhurst.....T	130.0	13.5	.....	4.0	.....	75½	1 230	24.453	213.9

* Steamed 21.54 knots per hour average speed for twenty-four hours. The I. H. P. would be about 20 700 to attain this speed.

N. B.—Class of steamer: M, mail steamer; C, cargo steamer; L C, lake cargo steamer; G, Government steamer; T, torpedo boat.

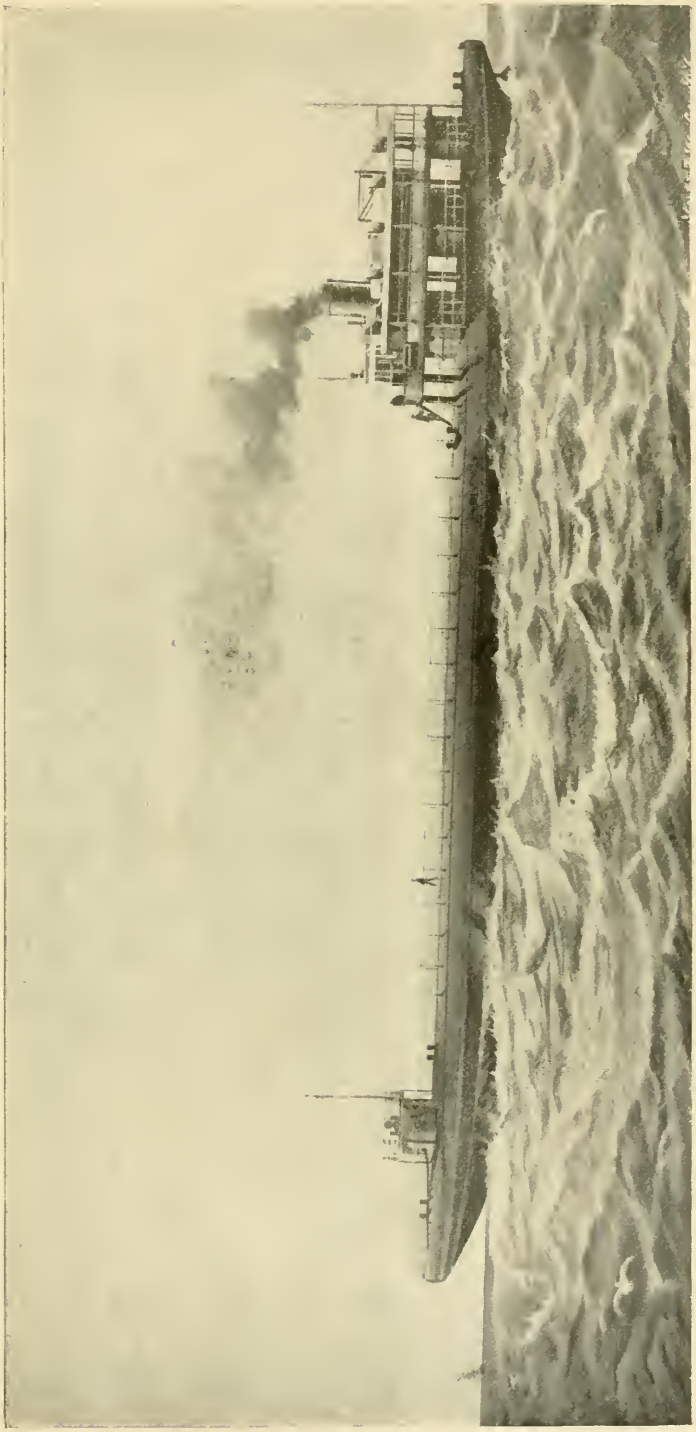
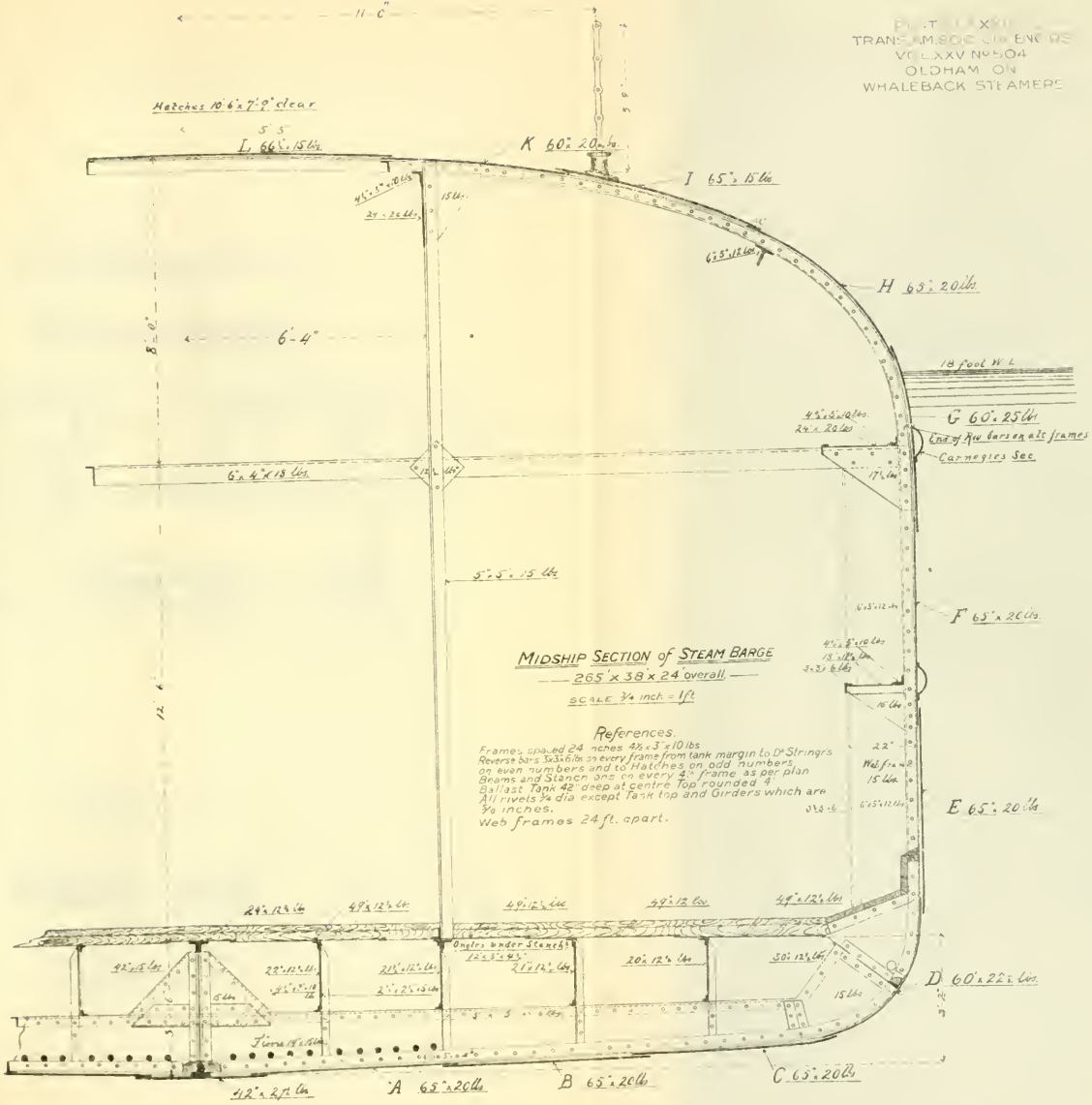
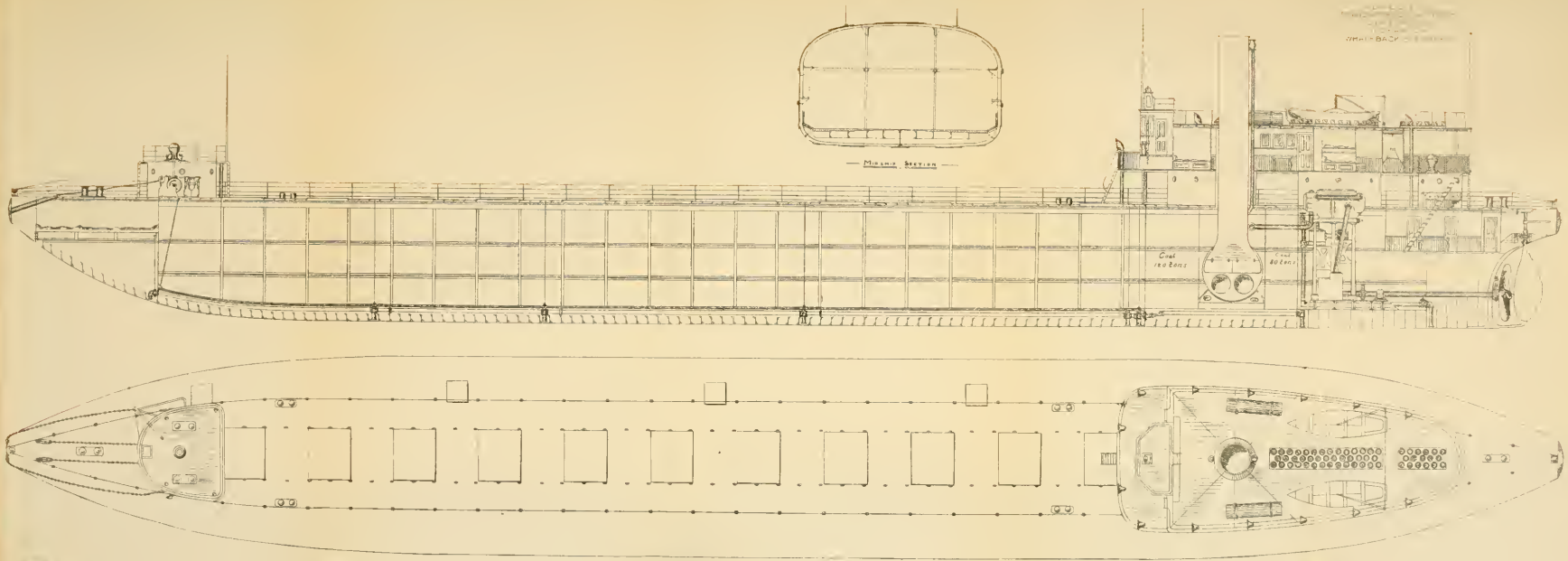


PLATE XXXI
TRANSAM. SOC. CIV. ENGRS.
VOL. XXV No. 504
OLDHAM ON
WHALEBACK STEAMERS




Midship Section
Coal 150 tons
Coal 60 tons

Also the following by Alexander McDougall, General Manager of the Steel Barge Company :

The *Colgate Hoyt* carries about 2 200 gross tons on 14 feet of water, and the consort referred to—107—carries 2 450 gross tons on the same draught, or each of them will carry 23 tons to the inch after that. Their maximum draught for good sea-going would be about 16½ feet. They are both the same size. The *Hoyt* has run 16 miles an hour the entire length of the lake several times and 12½ miles loaded. Her consumption of fuel and the good results obtained I think beat anything on record. Her engines are 26 and 50 x 42-inch stroke ; boilers, two 11 x 11½ feet—125 pounds steam.

DISCUSSION.

Charles E. Emery, M. Am. Soc. C. E.—To judge accurately of the performance of the vessels it is necessary to know the indicated horse-power and some further details of the engines and boilers. It is also desirable, from an economical point of view, to know the amount of coal consumed. Hoping to obtain more detailed information on this subject I wrote Mr. Frank E. Kirby, the engineer of the Detroit Dry Dock Company, and inclose his reply, which may be of interest.

My dear Sir :

Yours of the 23d inst. at hand. I have only been at home three days during the past two weeks. I regret that I am unable to furnish you any particulars of the whaleback engines, and there is so much misinformation about their performance that what is seen in the public prints is valueless. I have glanced through Mr. Oldham's paper. The high efficiency he claims for the *Sitka* and tow of whalebacks is true with tows of any kind of vessels.

With regards, I am yours truly,

Frank E. Kirby.

Herbert C. Felton, M. Am. Soc. C. E.—Referring to "Advance Copy" of Mr. J. R. Oldham's paper on "Screw Steamships and Tow Barge Efficiency, etc.;" the enclosed clipping, taken from the Philadelphia papers of to-day's issue in regard to the "whaleback" steamer *Charles W. Wetmore*, may be of some interest from a practical point of view as regards the cost of loading and discharging when in a port where suitable shore conveniences might be limited. In this case the dredge was moored between the pier and steamer, and, of course, occupied additional dock-room and consumed more time in handling freight.

"The work of loading the much-talked-of whaleback steamer *Charles*

W. Welmore was completed yesterday, and at daylight to-day she will begin her 13 000 mile journey to Port Townsend, Washington. Stevedores are not sorry the arduous task of loading the oddly-constructed craft is over. They say she is not constructed to carry cargoes of machinery, being without spars or any tackle by which the huge blocks of machinery can be hoisted into her capacious hold. It was necessary to secure the services of a dredging machine to load her."

Geo. S. Morison, M. Am. Soc. C. E.—I have nothing to say in the discussion, but I would call attention to the fact that the note as to the speed of the *Teutonic*, on page 12, is hardly correct. I was on board of her when she made her longest run, which was 517 knots from noon to noon in twenty-four hours and forty minutes, or a little less than 21 knots per hour. This would correspond better with the indicated horse-power actually developed.

J. A. Ockerson, M. Am. Soc. C. E.—Mr. Oldham's paper on "Screw Steamship and Tow Barge Efficiency on the Northwestern Lakes" comes at a very opportune time. The success of the "whaleback steamers" seems to have been thoroughly established and a new era in shipbuilding on the "Great Lakes" has begun. On a recent visit to these "inland seas," I was informed that several of the large shipyards were practically idle, as far as new work was concerned, awaiting the results of a thorough trial of the "whalebacks."

I hope the discussion on this paper will bring out more fully the details of construction of this new form of freight carrier.

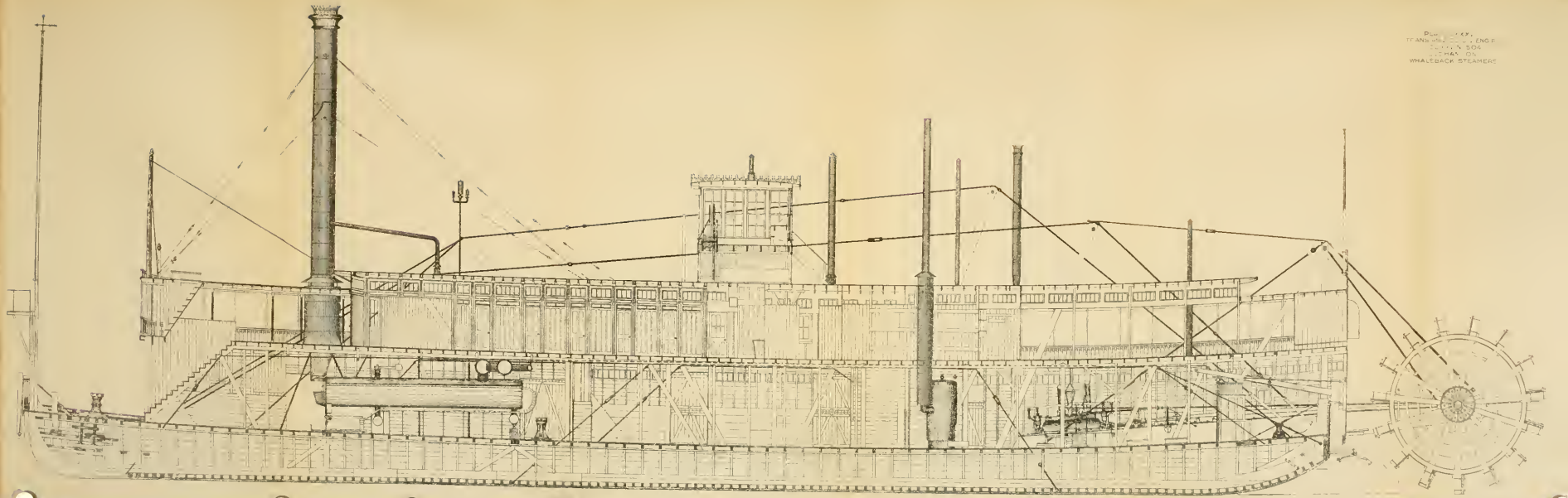
On the Mississippi River but little progress has been made in the methods of steamboat construction, except, perhaps, in the tendency to use steel instead of wood for the construction of hulls.

I take this opportunity to present a drawing of the Mississippi River Commission tow-boat *Minnetonka*, used in towing barges loaded with rock and other material from points above the Ohio River, to the improvement works carried on by the Commission on the lower river. (See Plate LXXXVI.) The general description of the boat and its machinery are as follows.

Length from forward nosing to splash bulkhead	176 feet 5 inches.
Length to end of fantails	202 feet 9 inches.
Width over all	35 feet 4 inches.
Width inside at top of floors	29 feet.
Depth of hold	5 feet 4 inches.
Crown of beam amidships	6 inches.

Draught.—When loaded with 1 500 bushels of coal the boat draws 4 feet 7 inches forward and 3 feet 4 inches aft, one-half of the coal being in the forward bunker.

Planking.—The bottom plank are 3½ inches thick, the knuckle 4 inches, and then planking becomes gradually thinner to the cutting



strake, which is 2¼ inches thick. The height between main and boiler decks is 11 feet 6 inches.

Boilers.—There are four return flue boilers 44 inches diameter, 26 feet 6 inches over the heads, made from steel $\frac{26}{100}$ inches thick, having 45 per cent. ductility and 70 000 pounds per square inch tensile strength, and each one has two flues of 12 inches and two of 14 inches diameter, and is double riveted throughout. There are two mud drums 15 inches x 16 feet 2¾ inches long. There are two steam drums 20 inches in diameter, and 16 feet 4 inches long. The steam pressure allowed is 165 pounds.

Fire Box.—The grate bars are 5 feet 2¼ inches long over all, by 3½ inches wide, with a middle slot 1 inch wide, and a maximum depth of 4⅜ inches; the tops are 20½ inches above the ash pan, and 17¼ inches below the bottoms of boiler shells.

The grate area is 81.75 square feet; the total heating surface 231.2 square feet; and total tube area 2 136 square inches.

Engines.—The main engines have cylinders of $22\frac{7}{16}$ inches inside diameter and 8 feet stroke. The steam ports to cylinder are 8¾ x 3⅜ inches, and exhaust ports $9\frac{3}{16}$ x 3¼ inches. The poppet valve is used for steam and exhaust. There are also reversing engines, electric engines and capstan engines.

Pumps.—There are the following pumps, viz., the "doctor," the donkey pump and the bilge pump.

Shaft.—The stern wheel shaft is made of iron, hexagonal in section, and its dimensions are 13 inches x 25 feet 10¾ inches. The cranks are 4 feet from center to center. The pitman is 38 feet 3½ inches long, a section at the middle being 7⅛ x 20 inches. It is built up of pine with iron straps top and bottom, 5 x 1 inches, bolted through with ¾-inch bolts.

R. H. Thurston, M. Am. Soc. C. E.—The new departure in ship construction, referred to in the paper of Mr. Oldham, is one of exceptional interest; but I think that it involves no new principles and develops no previously unfamiliar facts in the department of naval architecture and marine engineering. The experiment now being tried on so large and satisfactory a scale, simply illustrates the general principle that in marine engineering, when prompt delivery of freight is not important, the costs of transportation are made least by adopting a low speed. That this principle has a limit is probably very true, but this limit is found at some extremely moderate speed. When it is remembered that the power demanded to impel any vessel, at speeds for which its form is well adapted, varies as the cube of the speed, and that the cost in steam for work so done varies between the two termini of a given route, as the square of the speed, and inversely as the square of the time of transit; it becomes obvious that high speeds are enormously costly and that low speeds are correspondingly economical. Again, the modern steamship, as commonly built, is adapted for the best service in mixed traffic. It must be equally satisfactory as a conveyer of passengers and of freight,

and its freight may be either the so-called "fast freight," which can be assessed heavily for costs of transportation, or the freights of iron, steel, grain and cotton, which may just as well be transported at low speeds and delivered a comparatively long time after shipment. Such a vessel is adapted especially to neither one nor another of these diverse and contradictory purposes; and it consequently and naturally can do neither kind of work to best advantage and with maximum economy and profit to its owner. Build the ship for a single trade, and it becomes at once possible to attain previously unexampled economy.

Thus, the fast steamers in the transatlantic trade should be made passenger and "express" freight boats simply, and the McDougall steamer illustrates the same principle precisely, in being constructed in such manner as to be well fitted for exactly the reverse case, heavy freights at low velocities. Taking the speed of the former as 20 knots, its tonnage at its load displacement as 10 000, and its power as 2 horse-power per ton at its average speed; the same tonnage, in any hull equally well-shaped for the lower speed, could be driven at 8 knots by about one-sixteenth that power, or about one-eighth horse-power per ton, or 8 000 tons 8 knots an hour with 1 000 horse-power. At 7½ knots, the figure for power becomes about 600 horse-power, or a little better than is here reported for the *Colby* when towing "nearly 8 miles an hour" (7½ knots), and better than the *Sitka* is reported to have done. But the figures for the *Sitka* have no value, except as rough approximations, in the absence of any actual "indication" of the engines and a trial of engines and boilers. The rule for "nominal" horse-power gives no reliable gauge of the actual power of the machinery or of its efficiency or losses and wastes. The estimate of power demanded by the Rankine formula, based on ship resistances, is more likely to be closely approximate and may be taken as a probable result. It is not at all out of the way in itself, or as requiring any remarkable quality of boat to meet its figures. The co-efficient, 247, for the displacement formula, is not at all high; in fact, it is rather low for these times. The fuel account is good. The figure originally, I think, given by Mr. Forney, as representing the performance of good ships on the ocean, a half ounce of fuel per ton transported, per mile, at a 10 knot speed, is the equivalent of $\frac{1}{2} \div 16 \times 100 = 3$ pounds nearly, or less than that here given for the "whaleback," which exceeds the ocean steamer in consumption of fuel by 66 per cent. This standard is a remarkably high one, however, and the lake boat's performance is probably fully up to the average of ocean-going craft.

The self-trimming construction of these boats is a good illustration of the advantage which comes of the freedom gained by the naval architect, when he may design for a single and specifically characteristic work. The ordinary case is one in which decks for passengers and working decks for the men, must be provided. Here, discarding sail power, the "tumble home" of the old *Constitution* and her contemporaries of the

early part of the century is readily carried further, and far enough to permit this self-stowage of the grain which is expected to be the common freight of these vessels. The design seems to me to be an admirable illustration of a courageous and sensible working out of a specified problem, unhampered by custom or convention. This form of section also promotes seaworthiness, as claimed by the writer of the paper, and as does anything which permits the sea to pass over and by the ship, instead of breaking against her sides or on her decks. I was once for a year on duty on our largest and heaviest "monitor" iron-clad, built during our civil war, and I never spent a more comfortable year when at sea (and I have been in all kinds of ships, month in and month out, at all seasons on our Atlantic coast and elsewhere) than on the old *Dictator* monitor. I have known a claret bottle to stand by the hour (empty) on the mess-room table when a gale of wind was blowing overhead, the seas sweeping where they chose, but never causing noticeable rolling. In this ship there is, I presume, as there should be, some passage below decks, fore and aft, for stormy weather; and below decks, I should expect it to be, as claimed, thoroughly comfortable in the heaviest weather, and should anticipate, as a common result of the same formation of hull, impunity, so far as the ship is concerned, in passing though the heaviest gales. This point is, to my mind, certainly a good one, and a most important one. I like also the double bottom and water-ballast arrangement. I have experienced its good qualities in earlier years at sea, and am confident that it will prove satisfactory, if properly constructed and handled.

The seven conclusions with which the paper concludes seem to me to be thoroughly correct, and a most excellent statement of essential principles in this class of construction, and, in the main, for naval construction generally. The advantages which the oval section and ellipsoidal form of these vessels offer in providing opportunity for strengthening the upper line of the beam—for the ship is to be considered a beam subject to all sorts of vertical, transverse and longitudinal stresses as well—are well worth considering, as is their advantage in simplicity of construction, small cost and superfices, with large capacity and minimum skin resistance.

The one thing remarkable in this case seems to me to be the fact that shipowners and a naval architect have been found, having a sufficient independence and originality to work together in the solution of a simple obvious problem, unhampered by either precedent or apprehension of criticism, professional or unprofessional. I was attracted by the original designs of Captain McDougall, years ago, when he was endeavoring to introduce his fast ship with its singular section, and have been glad to see that he has taken up a more promising line of commercial work, and with such prompt success. As lake freight steamers, I imagine they will be found admirably suited for their work, and I shall not be surprised if they ultimately find permanent place in the transatlantic trans-

portation of slow, heavy freight. I would congratulate the captain and his friends on the success thus far attained.

CHARLES H. HASWELL, M. Am. Soc. C. E.—This paper is rendered interesting by the recital and some of the results furnished, and if I entertain the claims of the writer correctly, he advances several claims to superiority of this new type of vessel. Also, that this type, which is designated as "whaleback," affords "extraordinary speed," together with "large capacity and enormous dead weight ability," and that the speed of one of them is claimed as "somewhat phenomenal." Further, that they are exceedingly staunch, tow with ease and small consumption of fuel, roll easily, and that their stem is of an outline to produce the least possible destruction in the event of a collision. Now, in order to consider or concur in these very desirable qualities, one properly looks to discover the particular elements and features whereby their attainment, so boldly asserted, is supported, and to present this subject to those who are not familiar with this peculiar construction or type of vessel, I submit: In outline its submerged portion approximates to that of a semi-elliptic spindle, whilst the upper portion or freeboard has "tumbled-in" sides, with a curved flush deck and quadri-circular junction to the sides of the hull, with light iron stanchions, and wire rope or rods around the sides in place of a heavy rail and close bulwarks, without spars and rigging. As regards their alleged speed, there is not a single element furnished showing whereby greater effect is attained than there would be in a hull of the ordinary form, with like or equal areas of section, ease of lines and proportional power.

As to "capacity" and "dead weight ability," I fail to recognize how such a type of hull furnishes greater capacity, either for bulky cargo or dead weight; quadri-circular top sides not only do not present a single advantage, but in loading with light freight the space lost by such conformation of sides is lost to stowage. Neither is it shown how they furnish "capacity" in excess of any other vessel of the usual proportions and constructed of like materials; neither does any such advantage as it is claimed exist, as equal volumes give equal capacity and like weights like displacement.

Staunchness, which is also claimed for them, is dependent upon the proportionate dimensions of a hull and the integrity of its construction, and as this quality is attained in the very greatest number of vessels of ordinary design, I equally fail to recognize the justice of the especial claim submitted by the writer for this type over that of other unrigged vessels.

Concerning their navigation, it is also claimed that the absence of the ordinary top-gallant forecastle, deckhouses and like constructions on the spardeck, enables them to offer less resistance to wind and seas, and as a result their operation is more effective; on the other hand, I advance that the flush deck, protected only by an open iron or wire rail, renders

manual operations on deck in stormy weather wholly impracticable; and that erecting the wheel-room, cabin, berthing, galley and all deck requirements upon columns several feet above the spardeck, presents more effective resistance to the wind than the deck constructions of our ordinary freighting steamers; added to which the facility afforded to the boarding of seas and the retardation occasioned by their impact more than offsets the advantage claimed.

Inasmuch as there are not any elements given why they should tow with great ease and with greater economical consumption of fuel than other vessels, I am at a loss to consent to an operation that bears upon its face a negation of the axiom that like causes produce like effects. As the form of stem of a vessel is arbitrary with the designer, and as in practice it is presented in every practical outline, I am further at a loss to recognize any individual claim to the outline given or any advantage in a form of bow, not water-borne for a long distance, and hence subjected to the stress of the weight of anchors and chains upon it in a pitching sea.

Regarding their rolling qualities and stability, both of these operations depend upon form of immersed section of hull and location of center of gravity of the mass; and the conformation of one and the location of the other cannot be essentially affected by cutting off the upper corner of the gunwale. If the *Charles W. Wetmore* in her late voyage across the Atlantic in ballast had not had stowed the 600 tons of coal between decks she would have rolled and lurched to a degree that would have rendered her crew much pleased to make a port; hence that which is claimed as a merit was indispensable to safety of the vessel.

In conclusion, a review of all the elements submitted and claims advanced does not present or substantiate in any one instance an advantage over that of an ordinary and well constructed freighting steamer of like displacement and power, and alike without spars or rig; subject to the question, if such absence of spars is at all practicable with the requirements of safety of the vessel and crew.

Joseph R. Oldham, N. A.—With reference to the discussion on this paper I would say, that possibly in my effort to hit upon a terse title for my paper, I have made it obscure, for as I make use of the word "efficiency" in that title, I do not intend it to be limited to the expression for performance as regards horse-power and speed, but I use it in its more general or ordinary sense, as denoting general efficiency, with slight particular reference to seaworthiness.

In reply to your learned member, Mr. R. H. Thurston, I may say that there was no opportunity for actual "indication" of the engines and boilers of the *Sitka*. As to losses and waste, I freely admit that with such facilities as ordinarily exist on board of cargo steamers at any rate, that I lack the courage to assume what such losses may be, for I verily believe that marine engines will require much more careful indi-

cating than has as yet been done before I may know what the waste and losses amount to in the ordinary triple expansion engine; but then I am only a practical shipbuilder and marine surveyor, and my paper has no pretensions to being an exhaustive treatise either on the speed of screw steamships, or on the maximum power of marine boilers or power as indicated in marine engines.

My conclusions were based on certain practical results observed under ordinary conditions at sea, and my deductions give indications of maximum ability with regard to the working of common machinery in common use. The formula for horse-power is distinctively put forward as approximate, and I have found such very useful, when hurried estimates were required for steamers to carry a certain dead weight of cargo at a required rate of speed.

The co-efficient 247 may not be very high, but these are every day results with one fireman cleaning and feeding four large furnaces for six hours at a time, or with "watch and watch"; they are not trial trip figures, which obtain for a few hours only, it may be, and are never resuscitated, but as they are they indicate higher efficiency than the *Brunswick*, 234, *Queensland*, 203, at about the same speed, or the *Africa*, 173. Professor Thurston says my figures are "rather low for these times." I am rather of opinion that they may be low as compared with old times, for nearly all the old freight steamers I can call to mind were much finer models than the modern "tramp" steamer; and these, I think, would not compare unfavorably with the *Great Britain*, if similarly rigged, or the *Propontis* or *Delaware*—of course I mean apart from coal consumption. I sometimes imagine that our models, except in very high speed vessels, have deteriorated, like our poetry. I think it is hard to make good verse in these days, and with the conditions usually imposed on our shipbuilders, they say it is hard to make a nice model.

Allow me to say in reply to Mr. Charles H. Haswell, that the form of the *Whaleback's* bow is such that about 700 feet of surface is saved by cutting away the dead-wood or fore-foot, and this with but slight reduction in dead weight ability; and thus on the skin friction theory of resistance the *Whaleback* should steam as fast as a steamer of the same dead weight ability, with about 5 per cent. less propelling power; again, her extremely narrow stern may increase the speed co-efficient, and the unincumbered decks will do so in a head wind. Like weights certainly do give like displacements, but like displacements do not always give like dead weight ability.

These vessels up to the present time are of similar proportionate dimensions to the ordinary lake steamer, which admittedly and unavoidably has excessive breadth of beam, giving them too much stability of form. The bold tumble-home in the "whalebacks" reduces the length of the righting lever to a desirable extent, and thus lessens the shock at

large angles of inclination. The rolling qualities of ships are most certainly directly and severely affected by other portions of hull than the immersed section when upright.

Does Mr. Frank Kirby mean us to understand that in his opinion the ordinary lake tow barge will tow as easily as a "whaleback"? Then why are the latter being towed for about 25 per cent. less money? I would be pleased to concede the point of error claimed by Mr. George S. Morison in connection with the steamship *Teutonic*. I was not at sea in this case, however; I will gladly amend my figures if Mr. Morison will be a little more specific, so that my correction may be reliable. In conclusion I may state that I have never been commissioned by Captain McDougall or other officials of the American Steel Barge Company either to champion or defend their property. My paper was intended to be an impartial investigation into a novel type of steamer, and I trust that in that respect it has not wholly failed. It was my duty to survey and report on these vessels for, and to, the "Record of American and Foreign Shipping." After considerable anxiety I concluded that these peculiar vessels merited the highest class in the highest record of shipping, and so they were recommended and classed. Some people said that they would smother the crew, others said they would "turn turtle," others again prophesied all these calamities and much more. Now about the prophets. For three seasons these vessels have worked all over these lakes. One has crossed the Atlantic twice with impunity, another has "rounded the Horn," and many people are at this moment paying Captain McDougall the sincerest flattery summed up in the word imitation.

NOTE.—The author requests the following changes to be made on this page: Line 3, for "surveyor" read "engineer"; and in lines 23 and 24, for the words, "and these I think would *not*, etc.," read "and these latter I think would, etc."

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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(Vol. XXV.—October, 1891.)

THE SINGLE TRAP SYSTEM OF HOUSE DRAINAGE.

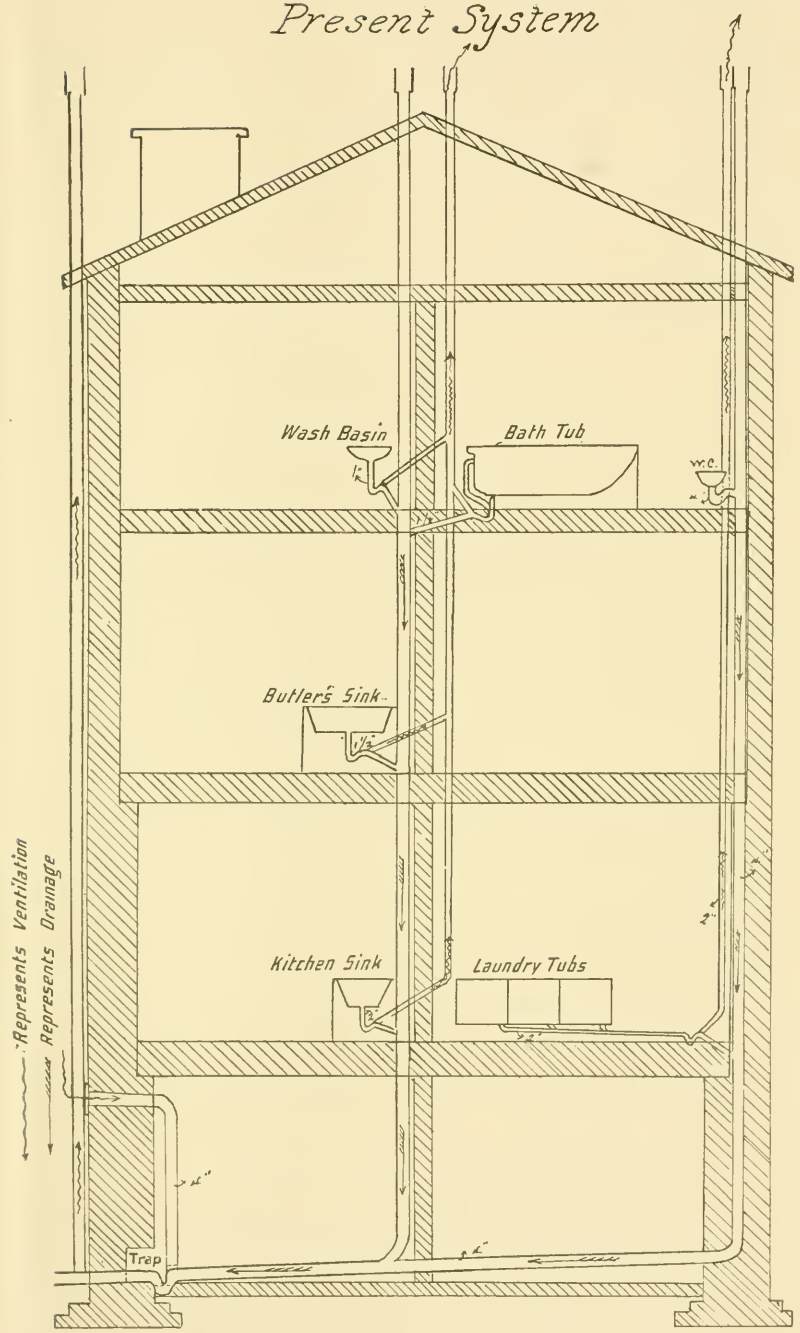
By LATHAM ANDERSON, M. Am. Soc. C. E.

WITH DISCUSSION.

The most approved system of house drainage now in use embraces the following features (See Plate LXXXVII) :

A.—A main trap in the sewer branch where it leaves the house, either immediately inside or outside of the outer wall.

B.—A vent the same size as the sewer branch, connected immediately outside of the trap, and extending either on the outside, inside, or within the house wall, above the level of the roof ridge. This outside vent is seldom applied or required by State laws or city ordinances; but is, in my judgment, the most important feature of the system. The objection urged against the introduction of this feature is, that in the case of the first house to introduce it, the sewers of the whole neighborhood will be vented at this point, and hence the said house will be plagued with more than its share of sewer air. It is true that under such conditions, there would be a constant rush of sewer air up the vent. But even so, the condition of the house would be improved. Such a vent could not fail to lessen the pressure of sewer air on the down-stream side of its own trap,


Present System
Wash Basin
Bath Tub
W.C.
Butler's Sink
Kitchen Sink
Laundry Tubs
Trap
Represents Ventilation
Represents Drainage

so that when water was passing through the trap, less sewer air would bubble back into the house. It is a prevalent popular fallacy that a water seal ever completely excludes sewer air.

Whenever water is discharged from a pipe or vessel containing air at a certain pressure, into a pipe at a higher pressure, air from the latter will invariably bubble back into the former, especially toward the latter end of the flow. This holds true whether a trap has been introduced or not. I have frequently observed this bubbling, even in a Bower's trap, which is closed by a rubber ball. Now such a vent as above described, must lessen the sewer pressure, and therefore correspondingly lessens, if not altogether prevents, such bubbling. At any rate, such a condition should not be allowed to occur for any length of time, because every house should be required to have such an external vent-pipe. Take the case of a solidly built city block, 400 feet long, with houses of 25 feet front, on each side, giving thirty-two houses in all. With a 12-inch sewer, 6-inch connections (as required in Cincinnati and most other cities), and 6-inch vent-pipes, the following ratio would exist between the sectional area of the main sewer and the aggregate area of all the vents. The area of the 12-inch sewer is 113 square inches; that of the vents 32 x 28¼, or 904 square inches; in other words, the total area of the vents would be eight times that of the sewer, so that with four house connections on each square, *i. e.*, two on each side of the street, the area of the vents would equal that of the main sewer. With an 18-inch sewer the area is 254½, or less than a third that of the vents. But in the opinion of many eminent engineers, 4-inch branches are much better for houses of ordinary size than 6-inch. With 4-inch branches and vents, the ratios would be as follows: For a 12-inch sewer as 113 is to 402, and for an 18-inch sewer as 254.5 is to 402. Hence it is apparent that under this plan, even with 4-inch house branches, it would be impossible for any abnormal pressure to exist in the same sewer. Even in the case of the 18-inch sewer, the venting would be ample, because it is doubtful whether any appreciable pressure ever exists in as large a pipe as this, in a properly designed system of sewers. The internal pressure is considerable only in the small pipes at the upper ends of the system. Hence, if these be thoroughly vented and ventilated, the larger and lower parts of the system will take care of themselves. It should be noted in this connection, that especially in long and large sewers, there is frequently a downward and outward flow of

air at the outlet; while there is invariably, as far as I am informed, a strong upward flow or pressure of air toward the upper openings or traps at the bottom of the system. The higher pressure at the latter points is increased and maintained by the inflow of hot water from baths, laundries, etc., which keeps the upper and smaller pipe sewers constantly at a higher temperature than the larger and lower parts of the sewer. Hence there must be some intermediate parts of the system where the sewer air is stagnant and without pressure, or even where the outer air flows into the sewer through the various vents. Another case is sometimes cited as an objection to outside vents. This is where a low house is hemmed in by much higher ones. Here, it is claimed, the air from the lower vent will ascend into the upper windows of the higher houses. This would undoubtedly be the case, if there were no vents on the upper houses; but with all the houses on the sewer outside vented, the preponderating upward flow of air must always be through the highest pipes—following a well known law of the flow of gases. Where the vents all act as down-casts (as above described), of course no inconvenience would result to either house. In some cases it might even happen that the lower shaft would act as a down-cast, at the same time that the air was ascending in the higher vents. But I freely admit, that in order to provide against all possible accidents, and to remove feelings of apprehension in the minds of the dwellers in the higher houses, the top of the vent of a lower house should never be allowed to terminate below the roof level of a higher one adjoining it. It could easily be carried along the wall of the higher house; as chimney flues usually are under like conditions. If the system of vents above described possesses the superior advantages herein claimed over any other system of sewer ventilation, it would be the height of folly to abandon it because of the slight and easily removed obstacles just specified.

C.—Another feature now required in all advanced practice, is a fresh air inlet carried from outside to a point just inside of the main trap.

D.—Every internal branch of the soil-pipe is carried full size, and as straight as possible, through the roof.

E.—Every fixture must have a separate trap of its own.

F.—In addition, every kitchen and butler's sink must have a grease-trap. From a sanitary point of view, this is one of the most essential features of house plumbing. Without it, grease from the dishes adheres to the inner surfaces of the pipes and especially of the sewer branch out-

side the house. This, in time, accumulates to such a degree that the pipes are frequently plugged solidly with grease and adherent trash, and they are always filthy and foul smelling to the highest degree.

G.—Every trap and dead space in water-closets must be separately vented at the top of the outer bend, the branch vents connecting with the main vent.

H.—A separate vent-pipe carried up as straight as practicable through the roof, and not, as is usually done in Cincinnati, into the upper part of the soil-pipe below the roof. The McClellan Anti-Siphon Trap Vent is designed to take the place of this complicated system of trap vent-pipes, and seems to answer the purpose admirably. It is approved by Colonel Waring, and the author is informed, has been accepted in lieu of vent-pipes by the sanitary authorities of New York and other Eastern cities. But, while it seems to possess decided superiority over the vent-pipe system, and to perform its functions perfectly while in order, he does not think it can claim to give absolute immunity from sewer air, because of the possibility of its becoming injured. For instance, the small pin which acts as a guide to the cup may be caused to move stiffly or to stick fast in its ring by corrosion or by the freezing of vapor attaching to it in very cold weather.

I.—Vent-pipes must never be carried on the inside of a fire-flue.

K.—Water-closets must never be flushed by direct connection with the water supply pipes, but must be provided with intercepting automatic tanks.

L.—House leaders for roof water, must not be connected directly with the sewer branch.

M.—All joints must be air-tight, and, of course, the materials and workmanship must be of good quality.

N.—In the best practice, all pipes are exposed and not sunk in the masonry, nor covered by plaster; being so arranged that they are everywhere accessible for examination and repair. Recently this has been made compulsory in some of the most advanced sanitary regulations.

The above provisions embrace the essential features of plumbing now required in the sewer connections of houses, exclusive of the water supply and of steam or hot-water heating.

Dr. E. S. McClellan, the inventor of the trap above mentioned, truly says, "Notwithstanding the commendable progress of recent years in house sanitation, the most approved methods of house drainage still have

defects, dangers and complications that call urgently for correction and simplification." If this be true of the most approved methods, the "defects, dangers and complications" of house drainage as it actually exists, in conformity with sanitary regulations, and as above described, are still more glaring. The following are some of its main defects:

First.—Considered as a system of ventilation it is complex, unscientific and uncertain in its action. One reason of this irregularity of action is that the air currents in the soil-pipe are descending, and would, therefore, be reversed in their direction by descending currents of water.

Second.—For reasons above described, but few, if any, houses are exempt from the entrance of sewer air. In cold weather this intrusion of sewer air is further facilitated by what has been termed "the house suction" of heated buildings. The place of air exhausted by chimney and other hot-air flues must, of course, be supplied by an equal amount of air entering through other channels. This causes the outer air to be sucked in at every crevice. According to Pettenkoffer this suction is so strong as to draw an appreciable amount of air through solid brick walls, except where they are painted, or the plastering is unusually dense. The colder the weather the greater will be the difference between the inside and outside temperatures, and the stronger the house suction. Therefore, under these conditions, more sewer air will be sucked into the house.

Third.—The numerous local traps form lodgments for filth. The constant presence of water in the traps through the house keeps the inside of the pipes continually moist, and therefore in a condition to promote organic decomposition and the propagation of germs. This is true whether the air in the pipes be stagnant or in motion; for, in the latter case, evaporation progresses rapidly from the water in traps. As a matter of fact, dampness and consequent foulness is the normal condition of waste-pipes, especially near their respective fixtures. It results from these conditions that by far the most filthy and noisome parts of existing sewerage systems are the plumbing pipes within houses, and, except where grease-traps are not used on kitchen sinks, the standing wash-basin is the nastiest feature of the plumbing. In improved appliances, especially in the simple outlet made by the Sanitas Company, the theory is that readily accessible parts will always be kept clean. In practice this cleanliness is more the exception than the rule, the appliances usually becoming and remaining foul after being in use for a short time.

Fourth.—The water in the traps left unused for a considerable time evaporates, thus destroying the seal. This would occur in the case of fixtures not regularly used—the basins in guest chambers, for instance, or throughout the house when it is left unoccupied for some time, as when the family are absent for the summer. Under such conditions, the rooms in question, or the entire house, become a main vent for the sewer; all the hangings, carpets, clothing in drawers and trunks, etc., being saturated with sewer air, provided the flow of the air in that part of the main sewer is toward the house ; *i. e.*, that the vents are acting as upcasts. The latter may or may not be more dangerous to health than the foul air within the house-pipes, according as there are or are not recent cases of infectious diseases in any of the houses using that part of the sewer.

Fifth.—The object of back-venting is to prevent the water in the traps from being forced or siphoned out by the discharge of other fixtures attached to the same soil or waste-pipe. If the pipes were always properly designed as to size, direction, bends and other structural details, such breaking of the seals could be effectually prevented by this contrivance. But this perfection is rarely attained in practice, so that it is probable there are few houses exempt from siphonage at some points.

Sixth.—Back-venting is conducive to one evil, so serious that many sanitary engineers and plumbers object to it altogether, preferring, as a choice of evils, the retention of stagnant, impure air within the pipes. This defect is, the increased rate of evaporation of trap water caused by the constant passage of air through the vent-pipes. According to Dr. John S. Billings, U. S. A., evaporation enough to break the seals "will occur" in two months, if the trap is not ventilated, and in about two weeks "if it is ventilated." Hence the evils mentioned under objection 4 would occur more quickly and surely in an unoccupied house with back-venting than without it.

Seventh.—In very cold weather the house air contains more moisture than that out-of-doors, even when the house is warmed by furnaces (convection) or steam radiators, in which case the air is hardly ever as moist as it should be. But even then, the warm house air usually contains so much moisture that its dew point is higher than that of the outer air. The upper ends of the metal vent, soil and waste-pipes being in the outer air, they will, of course, have the same temperature as the latter.

Consequently the watery vapor from within will be condensed on the inner surfaces of these pipes as it approaches their upper ends. When the outside temperature is very low this condensed moisture freezes, so that, in many cases, the pipes become completely plugged with ice, and the ventilation is cut off at the very time when the house suction is strongest, and when, as explained in No. 2, there is greatest danger from the entrance of sewer air. This condensation also aids in maintaining the internal moisture of the pipes noted above.

Eighth.—This tendency of sewer air to force its way into the house demands that every joint of the pipes shall be air-tight. This perfect tightness of every joint is rarely attained, if in fact it be attainable in practice. Even where the joints are air-tight when first made, there are several causes tending to produce subsequent leaks; among others, settlements of the building; injuries to the pipes by other workmen who follow the plumbers; unequal expansion and contraction of the pipes, especially where this is not properly provided for in the plans; repeated concussions or jarring of the floors and walls, especially in warehouses and buildings containing machinery; and the corrosion of pipes due to the condensation noted in No. 7. A case under this head recently came under the observation of the writer. A 2½-inch spiral riveted vent-pipe to a basin was carried up behind the plastering. The pipe projected slightly beyond the furring. The man who nailed on the laths, nothing daunted by this small obstacle, removed it by mashing the pipe, so that it was flush with the furring. This slightly cracked the pipe, so that corrosion by the water of condensation finished the job by boring a hole through the pipe. Through this orifice water trickling down the pipe flowed out in such quantities as to saturate the plaster and rot the lathing. The water was so contaminated by sewer air that it was as offensive to the smell as that from any other part of the plumbing.

Ninth.—The complexity of the system, and the extent of vent-piping required, renders it costly to construct and maintain as compared with a simpler plan.

Tenth.—The so-called "fresh-air" inlet, as usually constructed, is a burlesque on the name. "Foul" or "ground air inlets" would be a more accurate title. The outer opening is rarely more than a foot or two above the ground or pavement. Frequently it is let in flush with the surface of the sidewalk, and sometimes near the curb line.

As shown by the experiments and opinions of Dr. Angus Smith, Professor Ripley Nichols, of the Massachussets Institute of Technology; of Drs. Parkes and Bowditch, of Boston; Buchanan, of England; Pettenkoffer, Simon (the noted English sanitarian), and many other scientists—"ground air," *i. e.*, air within the ground and immediately above its surface, is much more impure and injurious to health than that obtained even a few feet higher. Hence, it is radically wrong to place a fresh-air inlet on or near the ground.

Mr. Richard Murphy, a plumber, of Cincinnati, has devised a system of plumbing intended to remedy some of the above defects of the existing system. The object especially sought is to cause the air and water currents to pass in the same direction within the pipes. This is effectually secured, under favorable conditions, by the following simple and ingenious device. From a point in the soil-pipe, just above the main trap, a vent-pipe is carried to a chimney flue and carried thence inside the flue to its top. The other (local) vent-pipes are carried out through the roof, as in the present system. In order to prevent any of these from becoming up-casts, the tops are throttled, so that the aggregate sectional area of all the inlets shall be only equal to that of the main vent-pipe. All local traps and other fixtures are as in the prevalent system. When properly designed and executed, this plan certainly has the following merits, as claimed by the inventor (See Plate LXXXVIII):

First.—The traps are not so likely to be siphoned, nor forced by increased back pressure resulting from suddenly reversing the upper air currents in the pipes.

Second.—In the present system the upward pressure of the ascending air obstructs and retards the discharge of the liquids by the pipes; whereas, in his method, the descending air current assists, to some extent, the discharge.

Third.—In the existing plan, the ascending current of air aids capillary attraction in retaining moisture on the surface of the pipe, while a descending current assists the force of gravity in carrying the adherent moisture down the pipe. Moreover, the air introduced from above must always be drier than that ascending from the wet soil pipes below. For these reasons, the pipes in Mr. Murphy's system would always be drier, and therefore less injurious to the inmates of the house.

Fourth.—The tendency to suck the air in at the fixtures instead of forcing it out into the room, is stronger in Murphy's plan.

Fifth.—In the upper floors especially, the air, as it passes the traps, is much purer than if drawn from the soil-pipes below.

Sixth.—Considered as a whole, the plan of ventilation is simple and effective, and is based on a sound scientific method. As far as the author is informed, this cannot be said of any plan of house-drainage ventilation now in vogue. If such a complex and irrational method of ventilation as any of these latter were attempted in deep mining, the miners would certainly be suffocated. But while this system, as described and illustrated by its author, has the merits above claimed, it fails to remedy any of the other defects enumerated above. Besides, it has the following defects of its own:

First.—The main vent is carried inside of a smoke-flue. This is a glaring defect, and should never be tolerated for the following reasons:

A.—The pipe is liable to corrosion from the smoke and the joints to become loose from excessive expansion and contraction.

B.—It is inaccessible, so that when repairs are necessary they can only be made at great cost and trouble, preventing at the same time the use of the chimney during repairs.

C.—If the chimney were to be disused for a time, and the pipe leaked, the foul air would find its way into the house.

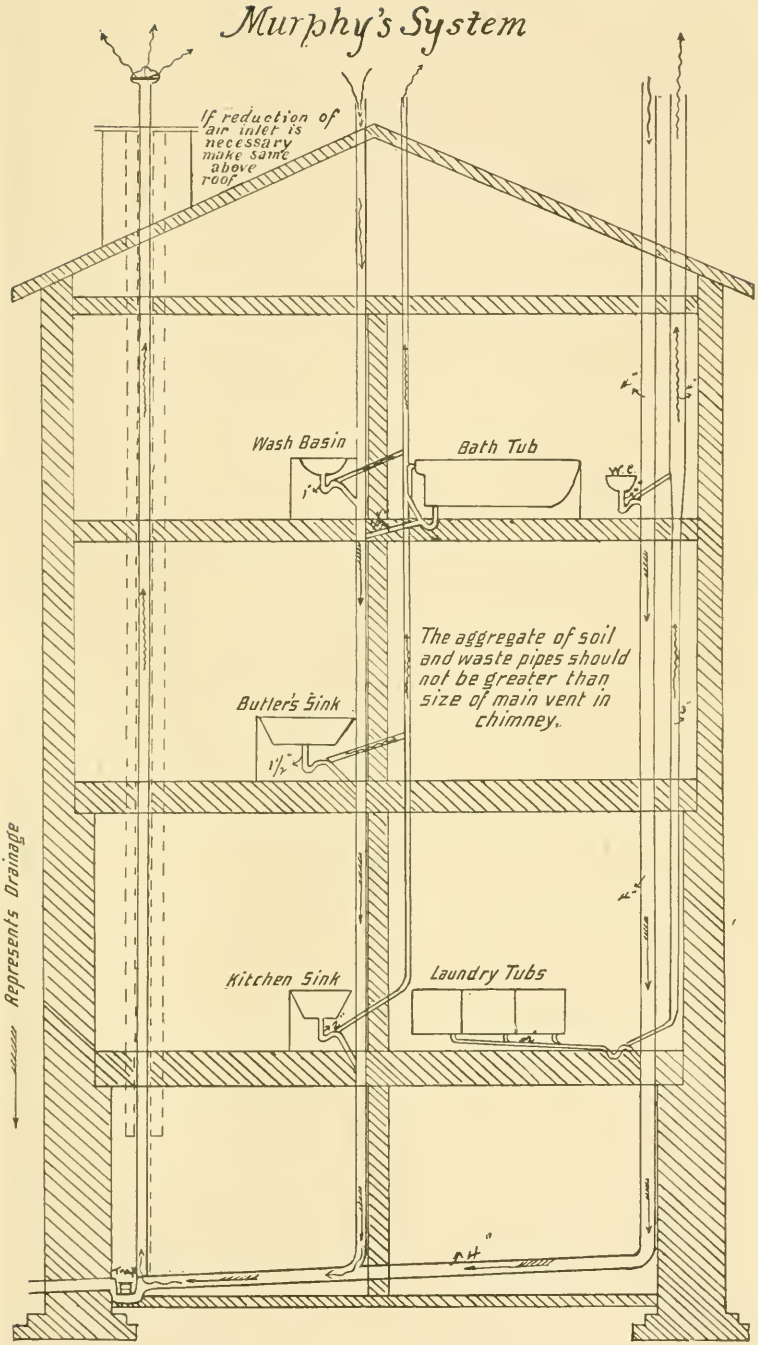
D.—When so cooled the pipes would be liable at times to act as a down-cast shaft. In which case the old system would be restored.

The main vent would only be effective as an up-cast when the temperature of the vent was so much higher than that of the outer air that the resulting differential pressure or head would be sufficient to overcome the frictional resistance of the pipes. In the case of a flue with an intermittent fire, that of the kitchen chimney of a dwelling for instance, it would often happen that this favorable ratio of temperature did not exist. A smoke flue could only be safely used to assist ventilation by carrying the vent-pipe outside the flue in contact with the heated brick-work, the pipe being everywhere accessible.

Second.—But the plan under consideration retains the most objectionable feature of the old, viz.: the retention of water-traps, the reservoirs of filth and moisture—a constant menace to health. No pipes can be kept reasonably dry or clean which contain these parts.

Third.—It does not provide for the outer or sewer vent.

Fourth.—In very cold weather, especially in upper rooms, the traps would inevitably freeze solid.


Murphy's System
If reduction of air inlet is necessary make same above roof
Wash Basin
Bath Tub
W.C.
The aggregate of soil and waste pipes should not be greater than size of main vent in chimney.
Butler's Sink
1½"
Kitchen Sink
Laundry Tubs
4"
Note
Represents Ventilation
Represents Drainage

To obviate, as far as practicable, all the defects above described, the following plan, called the single trap system, is proposed, of which the essential features are (see Plate LXXXIX) :

A.—The abolition of all local traps within the house, except the grease-traps to sinks, which are so arranged as not to be air-traps.

B.—The use of a single trap between the sewer and the house, an essential feature of the trap being a metal flap-valve just below it.

C.—A main up-take house vent, situated as in the Murphy system, immediately above the main trap. But it is essential to the proposed system that the upward current be unintermittent, and that the means provided for this end be simple, inexpensive and absolutely controllable by the occupants of the house.

D.—There are no inlets for outer air. The only air drawn into the pipes is that from the rooms, through the open ends at the various fixtures.

These are the four distinguishing features of this system of house drainage, and it is readily seen that it is a radical departure from present practice. The following are the merits claimed for it :

First.—The ventilation is simple and effective.

Second.—Cleanliness and consequent healthfulness. There being no standing water anywhere within the pipes, and the warm, dry house air constantly descending in them, their surfaces would be thoroughly dried and aerated within a few minutes after each using. Hence, dryness would be their normal condition. The germ-breeding slime which now coats our pipes could not form, being burnt up by the passing dry air. In other words, the drain pipes inside the house would become the least filthy and dangerous part of the sewage system; whereas, they now constitute the most dangerous and deadly part.

Third.—The dangerous air of the outer sewers is effectually excluded from the house, a result never heretofore fully attained.

Fourth.—By the universal use of the outer or sewer-vent the ventilation of the main sewers will be as near perfection as, in the judgment of the writer, it is practical to attain, and vastly better than by any other plan heretofore attempted or devised.

Fifth.—Inasmuch as the air must always be sucked into the open ends of the fixtures in the rooms, the drain pipes aid in the ventilation of the house, instead of being the main source of contamination of house air as at present.

Sixth.—The foulest parts of the house drains are the small pipes immediately below wash-stands. Under this system, these parts may be readily cleaned out by brushes attached to wires and fitting the bore, as in cleaning gun barrels.

Seventh.—General simplicity of construction and accessibility of parts.

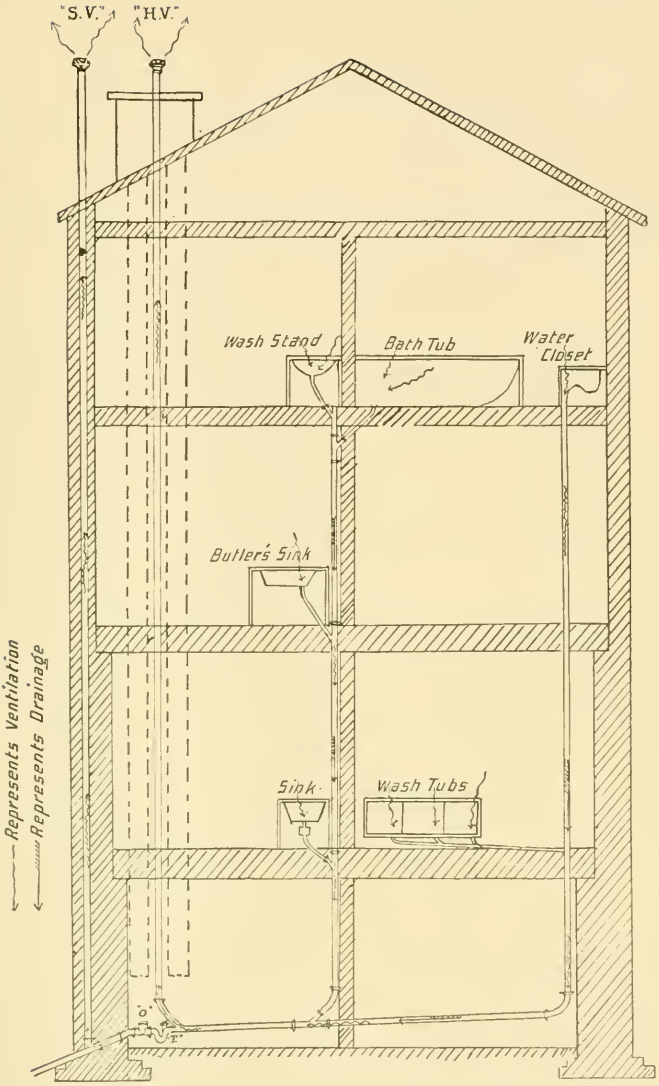
Eighth.—Owing to their cleanliness and healthfulness the various fixtures may be placed anywhere within the house without offense to the senses or danger to health, thus materially facilitating the planning and arrangement of houses in regard to the plumbing. Under the present plan, in order to avoid the serious and only partially avoidable danger to health caused by "modern conveniences," it would be good practice, from a sanitary stand-point (though rarely followed by architects), to plan the plumbing and build the house around it. Under the proposed system the plumbing could readily be adapted to any house plan.

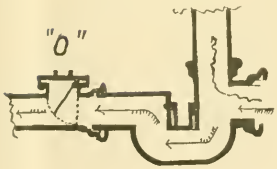
Ninth.—Inasmuch as sewer air is effectually excluded, and that in the house pipes purified, any air leaks that might exist in the pipes would not prove injurious to the health of the inmates. In view of the difficulty or impracticability of always securing air-tight joints, the importance of this ninth claim is manifest.

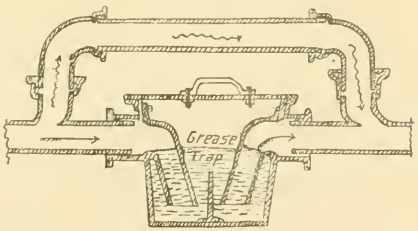
The most frequent objection which will probably be urged against this radical change in house drainage is that it is not automatic, but that it demands, as applied in most cases, the constant supervision of some inmate of the house, for instance, to see that the gas jet, where that is used, is kept lighted; and that where the means for forcing the upward draft in the house-vent cease to operate, the vent may become a down-cast and the air in the soil and waste pipes be discharged into the house. To these objections it is answered:

First.—That the present system, while pretending to be automatic, utterly fails in its object of securing a "dulce domum"; that the failure to supervise and cleanse its complicated and usually inaccessible parts adds greatly to its noisomeness and danger; and that the prevalent idea that it is automatic encourages this neglect and uncleanliness.

Second.—If, in the proposed system, the means for maintaining a forced draft were neglected, and, in exceptional cases, the long house-vent were to become a down-cast, the air thus admitted to the house would traverse the pipes, which, in comparison with the present wet ones, abounding as they do in miniature cess-pools, would be dry, sweet







Section Showing Connection of Grease Trap without intercepting Ventilation.

and clean. If the pipes were sufficiently coated to smell offensively, notice of the derangement of the ventilating appliance would immediately be given to the inmates, and the evil would be at once corrected. The presence of such pipe air could not possibly be fraught with as much danger to health as it would if it emanated from the present stinking plumbing fixtures or from the main sewer. In this connection, it should be noted that, as already explained, if every house were furnished with a sewer-vent, back pressure by sewer air on the main trap would be an impossibility; and that this contingency is still further provided against by the metal flap-valve. Should a house be left unoccupied for some time, and the house-vent act as a down-cast, the pipes would become thoroughly dry and aerated after the first few hours, so that they could not furnish during the entire period of vacation an unintermittent supply of foul air, such as exists in a house acting as a sewer-vent for the whole neighborhood.

The most active opposition to this reform in plumbing would probably come from the plumbers, under the idea that the amount of work required from them in order to supply a given number of fixtures would be so greatly diminished. But this is a short-sighted view. The house owner has a certain amount which he is willing to expend. If, under the new system, the plumbing for each fixture costs less—then the owner will usually add more "modern conveniences"; or, if he be an American, and has the same passion for pretentious, gaudy ornamentations as most of his fellow countrymen, he will spend the balance in polished marble, nickel or silver plate, or some other costly and shining adornments. In any event, the business of the plumber would not be curtailed.

It is admitted that, in the proposed system, the maintenance of a forced draught would usually demand the watchful attention of some human being, and that the outlets from the wash-basins and baths would not automatically cleanse themselves any more than the basins, or than any other domestic utensil. But, if the system possesses all the advantages here claimed for it, the small amount of care and labor required to insure its operation and to keep the parts clean, is a small price to pay for exemption from the formidable and protean evils of modern plumbing. In order to insure the successful operation of this plan, it is essential that the flow of air shall always be maintained in the proper direction, and that the means taken to secure this result be simple, inexpensive, durable and always under the control of the occupants. As upon this

the success of the plan mainly depends, it is the special object of this paper to prove to sanitary engineers that this is practicable and easily attainable.

The forced upward draught in the house vent may be secured by any one of the following means, viz.:

A.—In houses where there is a chimney flue in which a fire is constantly maintained day and night,—as in hotels, flat buildings, factories, etc., the vent is run up in contact with the heated masonry of the flue.

B.—A gas jet may be inserted in the vent at any point of its course and kept constantly burning.

C.—In houses where steam or hot water is constantly used, a short section of the vent-pipe may be kept hot by passing a few coils of steam or hot-water pipe around the vent and in close contact with it.

D.—Where hot air is constantly used for drying or heating purposes, a section of the vent-pipe may be boxed in so that a current of heated air shall pass around it.

E.—A small chamber may be built around the trap and lower end of the vent-pipe, in which chamber air may be kept heated by a small base burner stove, in which fire is constantly maintained.

F.—Where steam, electric or water-power is constantly employed, the draught may be maintained by means of a fan or an air-pump of any kind which may act, either by suction or by injecting the air into the lower part of the pipe.

G.—In buildings ventilated by the vacuum movement, either mechanically or by an aspirating chimney, the vent-pipe should be carried directly into the ventilating flue. Where the mechanical movement is used, a small branch may be led from the force main supplying an upward air jet in the vent-pipe at any point in its course.

Plan *A* should never be solely relied upon, because any chimney is liable to be out of use occasionally, owing to accident or for the purpose of repairing the furnace or machinery. Where the chimney is used, it should only be as an adjunct to some one or more of the other means indicated. In fact it is safer not to rely on any one means. At least two should be used in conjunction, so that should one fail the other may be available. In dwellings especially, the kitchen chimney as a sole means is unreliable and is becoming more so; for the time is not far distant when gas will replace solid fuel for cooking and other domestic uses.

It is well to call attention to one result of the omission of local traps which, at first sight, might be considered a serious defect in the system. When a fixture in the upper part of a house is emptied, the sudden rush of water in the upper part of the pipe may tend to force more or less air out of the openings in the lower fixtures. In the case of wash-basins and baths this may be effectually prevented to any objectionable degree by throttling the outlet so that the plug has less area than the waste-pipe. The same plan should be used for water-closets, the branch should be of 3-inch pipe discharging into a 4-inch soil-pipe. Two and a half inches is shown in practice to be large enough for the branch; hoppers with that size discharge being successfully used in insane asylums.

DISCUSSION.

Charles B. Brush, Director Am. Soc. C. E.—It seems to me that the plan recommended by the author is open to criticism. The worst gases that we have to contend with are those which generate in the houses rather than those from the sewer. The sketch marked Fig. 3 seems to indicate that, in the writer's judgment, a trap should be maintained on the sewer at the entrance of the building. It is proposed to protect this trap with two shafts, running above the roof in front of each building. There is no other provision made for ventilation of the sewer pipe, and no other traps are permitted. No other provision has been made for carrying off the gases from closets and sinks. The sketch, Fig. 1, provides for traps and for their ventilation. The fresh air intake is at the entrance of the building. The other sketch seems to be the same as the "present system," with the exception that the fresh air flue runs up to the top of the house. I doubt very much the necessity or the desirability of this trap at the entrance of the house. I think it is better to allow complete circulation to take place, by providing a fresh air vent to the roof from each trap. The criticism of the author in relation to the fresh air intake is good. The better way to avoid complications is to abolish the trap in front of the house. This trap is always filled more or less with grease and the deposits which come through the pipes. Finally the trap chokes and then there is no escape for the gases except into the houses.

The author criticises what he calls the "present system," because it is not automatic. From time to time this system will have to receive attention. So will any system. The principal attention this one will have to receive is simply to see that the basins are used. The water will have to be turned on, and as soon as this is done the trap will be filled. I cannot imagine any attention that can be more readily given. Every

intelligent householder ought to know that the water in each trap should be renewed at least once in every two weeks. If there are air vents from the traps it will be almost impossible to empty them and thus break the seal. The basins ordinarily in the living rooms of the houses, the kitchen, etc., are used so constantly that there is no question about these traps. The question arises principally in relation to the guest chambers, and to the house after it has been closed awhile. Common prudence would teach that under these circumstances it is better to turn on the water and let it run for a time before using rooms that have not been occupied for some time. I think it is dangerous to rely on any system such as is proposed in Fig. 3 of this paper.

Latham Anderson, M. Am. Soc. C. E.—It seems to the author that Mr. Brush has failed to understand fully the essential points of difference between the present system and the one proposed. The essential features of the latter are : 1st, the complete isolation of the air in house plumbing from that in the sewers—thus constituting in effect two distinct systems of ventilation ; and, 2d, the most efficient ventilation of each system by the simplest practicable means. Now, a trap of some form is the only means yet devised of separating the two systems, as far as the author is at present informed. The use of a cut-off trap between the sewer and the house, with an external vent from the sewer extending above the roof, is not a novelty. It is a feature demanded by many of the most advanced sanitarians of to-day. The use of the external trap, either with or without a sewer vent, is almost universal. Therefore, all of Mr. Brush's arguments against the use of this trap apply as well to the existing system as to that proposed by the author.

Mr. Brush says, "It is proposed to protect this trap by two shafts, running above the roof in front of the building. There is no provision made for ventilation of the sewer pipe, and no other traps are permitted. No other provision has been made for carrying off the gases from closets or sinks."

The statement that both vent pipes are designed to "protect" the main trap evinces a misconception of the author's design. The inner vent pipe is not intended in any sense as a protection to the said trap, but only as a means of ventilating the house system. As to there being "no other provision made for ventilating the sewer pipe"—what existing system has as much ? Surely Mr. Brush could not have read that part of the argument showing the excess of area of the vent pipes over that of the sewers themselves, in all sewers of 18 inches diameter and less.

The statement that "No other provision has been made for carrying off the gases from closets or sinks," implies that, in Mr. Brush's opinion, this means is inadequate. Fortunately it is not necessary to discuss this on abstract principles, as an untried experiment. Scharn Weber's patent of a gas jet burning in a vent pipe has proved a complete

success in maintaining a constant circulation and an entire absence of odor about the fixtures. Urinals attached to the Smede-Ruttan system of heating and ventilation are notably well ventilated and free from odor. In the Ortiz flat building, in Cincinnati, a forced upward current is maintained in the vents by having the lower ends of the latter open into the cellar and carrying a single return of steam heating pipe a few feet up the vent shaft. Neither is the downward draught in the local vents any longer a matter of experiment. Mr. Murphy has demonstrated its practicability and its benefit in the last-named city, by applying it in practice. Carrying the air currents through the soil and waste pipes in the same direction as the liquid flow, is the distinctive feature of Murphy's system. This fact seems to be entirely lost sight of by Mr. Brush when he says, "The other sketch (Murphy system) seems to be the same as the 'present system,' with the exception that the fresh air flue runs up to the top of the house."

Referring again to the main trap, it is said, "This trap is always filled more or less with grease and deposits which come through the pipes. Finally the trap chokes and then there is no escape for the gases, except into the houses." Do not the same objections hold with regard to all the other traps in the house? And is not a small trap under a low scouring head more apt to clog up than a 4-inch or 6-inch trap under the high fall usually obtaining at the main trap? As to obstruction by grease, it is the author's experience that straight pipes anywhere below the kitchen or butler's sink are more apt to become plugged by grease than are the traps. The presence of kitchen grease in either is inexcusable, since a good grease trap so completely prevents it. A properly designed trap should be so located (preferably within the house) as to be readily accessible, and should have a hand-hole for cleaning out obstructions.

The author's criticism as to the non-automatic character of the present system is also misunderstood. It is meant that householders generally seem to regard it as automatic, when reality it is not. The author agrees with Mr. Brush that every system requires attention, the one proposed not being claimed as automatic. That system is the best which is simplest and has the fewest points requiring attention. In the proposed system there are three, viz.: the main trap, the grease trap, and the appliance for producing a forced draught. Furthermore, the occupant of the house will be compelled to attend to the first two of these appliances, for if he do not, the system will not work at all. As far as the main and grease traps are concerned, this is true of any system of plumbing; so that the ventilating apparatus is the only feature to be looked after, and, where constant steam service is used, this becomes automatic, as at the Ortiz flats, above noted. Mr. Brush adds: "If there are air vents from the traps it will be almost impossible to empty them and thus break the seal." The author must take issue with this statement.

Exhaustive experiments made by Dr. Fergus, of Scotland, upon vented traps conforming in every respect to existing sanitary laws showed that they are not an efficient safeguard against the siphoning or forcing of traps.

Quoting again from the "discussion":

"The question arises principally in relation to the guest chambers, and to the house after it has been closed a while. Common prudence would teach that under these circumstances it is better to turn on the water and let it run for a time before using rooms that have not been occupied for some time."

If in any occupied house or room the traps become unsealed by evaporation, it is common experience that the house suction makes the house the main vent for the sewers of the neighborhood. If this sewer air should contain any noxious germs, the carpets, hangings, bedding—even clothing packed in drawers and trunks, becomes saturated with the vitiated air. Under these circumstances, if the Niagara River were emptied through the traps, it does not appear that the above named textile fabrics would be disinfected thereby.

Finally, the following sweeping conclusion is arrived at: "I think it is dangerous to rely on any system such as is proposed." But the dangers are not specified. Is it the danger of the entrance of sewer air into the house? That is obviated by destroying all back pressure from the sewer by the numerous house vents, as explained in the paper, and by back pressure valve. Where a house is to remain unoccupied for some time the breaking of the seal could still further be guarded against by covering the water in both branches with a layer of glycerine. This would retard the evaporation of the water for months.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—October, 1891.)

THE PROPOSED LAKE ERIE AND OHIO RIVER SHIP CANAL.

By JOHN M. GOODWIN, M. Am. Soc. C. E.*

WITH DISCUSSION.

The General Assembly of Pennsylvania at its session of 1889 authorized the appointment of a commission to "determine the feasibility of connecting the waters of Lake Erie and the Ohio River by a ship canal;" and, in event of finding such a connection feasible, "to lay out a route for such canal," and make an estimate of the cost of construction of the waterway. The commission on this subject, appointed by Governor Beaver in accordance with the terms of the joint resolution above quoted, entered upon the duties with which it was charged in the spring of 1890, and in February, 1891, rendered to the Legislature a report declaring the construction of a proper ship canal, to afford navigation for large vessels between Lake Erie and Pittsburgh Harbor, to be entirely feasible; and estimating the cost of such canal (with locks 300 x 45 feet in the clear, with 15 feet of water on the miter sills), at $27 000 000. The commission printed an edition of five hundred copies of its report; and upon filing said report, placed in the hands of the State Printer photo-

* This paper was given in outline by Mr. Goodwin, who was a member of the commission, at the Convention held at Lookout Mountain. Mr. Goodwin died October 27th, and he had not read the final proof of the paper.

engraved plates of the profile of the canal route, and of nine maps and one lock diagram prepared by the commission for illustration of its work.

Upon receipt of this report the Legislature adopted a resolution directing the printing, by the State, of three thousand copies of the report and the accompanying maps, for use of the commission. At the suggestion of the commission, it further instructed the representatives of Pennsylvania in Congress to take steps to secure a review of the work and findings of the commission, by officers of the U. S. Engineer Corps; and directed the commission to submit to the Legislature a supplementary report to be made after the rendition by such engineer-officers of their report upon the review sought, as aforesaid.

In February of this year the commission placed in the library of this Society a copy of its report, rendered as above noted; and now presents for the inspection by the convention several maps, printed from the plates aforesaid,* showing, respectively, the country between Lake Erie and Pittsburgh, with the projected route of the ship canal, and the location of the canal through each of several of the larger manufacturing towns of the Shenango Valley. These maps the commission delivers to the Secretary of the Society for deposit in the Society's library.

Circumstances do not favor the presentation at this time of any extended remarks upon the ship canal project in question; nor, indeed, were ample opportunity offered, could the author do more than review the matter in this regard presented by the report of the commission, which quite exhaustively treats its subject—considered with relation to the engineering features of the enterprise and to the circumstances in view of which the commission adjudges the Lake Erie and Ohio River Ship Canal a commercial necessity.

The commission claims that the report fully demonstrates the feasi-

* List of maps prepared by the Canal Commission. 1. General map of country between Lake Erie and the Ohio River, from the Allegheny River on the east to longitude of Ashtabula Harbor on the west, with route of canal, etc. 2. Map of Conneant Harbor. 3. Map showing location of ship canal through Greenville, Pa. 4. Map showing location of ship canal through Sharpsville, Pa. 5. Map showing location of ship canal through Sharon, Pa. 6. Map showing location of ship canal through Newcastle, Pa. 7. Map showing location of ship canal through Rochester, Pa. 8. Sketch of route of canal in vicinity of Conneant Aqueduct. 9. Map of right bank of Ohio River, from Davis Island Dam to Rochester. 10. Diagram illustrating plan for passing vessels from ship canal through Davis Island Lock into the Ohio River. 11. Profile of canal route, from Conneant Harbor to the Ohio River.

Maps not prepared by the commission: *A*. Map of the Monongahela River. *B*. Map of country between Pittsburgh and Lake Erie, 1824.

bility of constructing the canal, and clearly illustrates the manner in which and the extent to which, the canal will facilitate and cheapen transportation between the ore, grain, and lumber-producing Northwest and the iron-making and coal-producing regions of Eastern Ohio, the upper Ohio Valley and Western Pennsylvania. It presents, likewise, thoroughly authenticated facts fully supporting the finding of the commission that the construction of the ship canal in question is the only means available for that enlargement and betterment of existing transportation facilities between Lake Erie ports and the iron-making districts of Eastern Ohio and Western Pennsylvania, which is absolutely necessary to the maintenance of the present rate of industrial progress of those districts, as well as to the continued development of the mineral resources of the Northwest, and an adequate and otherwise satisfactory supply of the Northwest with fuel.

Of the various considerations presented in the report arguing the propriety of the adoption by the nation of the measure of building the Lake Erie and Ohio River Ship Canal, none are more intrinsically important than those relative to the necessity of the canal as a means for ready defense of our lake ports, and of the great fleets navigating our inland seas by authority of and under what ought to be the sure protection of the United States of America.

With regard to suggestions that the canal will best be constructed so that, if need be hereafter, the available depth of the waterway may be made about two feet greater than that for which the commission has planned, the report says: "The questions raised by this suggestion will, of course, have due consideration by those who, eventually, will determine the course preferably to be pursued in this and other like directions."

The discussion of this question of the appropriate depth for this canal involves the consideration of many and very widely comprehensive fields of inquiry. The author will only say now, that the commission, after duly examining the matter, sees no sufficient ground for modifying its recommendation that the canal be made with a bottom-width of 100 feet and depth (of water) of 15 feet.

The author trusts that he may soon be able to supply copies of the report of the commission, and this will give a complete exhibit of the facts upon which the conclusion is reached that the canal is a "necessity."

To make a very brief statement of the project, it is to carry a canal from Lake Erie, at a point just west of the State line between Ohio and Pennsylvania, to the Ohio River at Davis Island Dam. To bring the canal to the Ohio River merely would be money and time thrown away, because there would be no inducement for vessels to come through if they were landed in the Ohio River without means of getting to the City of Pittsburgh. For that reason the commission decided to continue the canal to Davis Island Dam, and there make the connection. With that connection there would be a depth of water from Lake Erie to the harbor of Pittsburgh. There was a canal which connected Lake Erie with the Ohio River, extending from Erie Harbor to the Ohio River at a point 26 miles below Pittsburgh. In consequence of this termination, so far short of its proper objective point, the canal had very little commercial value and was of very little use. If it had not been for the diverse interests of the railways it might, however, have served a better purpose than it did. The distance from the lake to the Ohio River at the mouth of the Beaver River, by the route of the canal, is 103 miles; it is only a few miles greater than that on an absolute air line. The point where the proposed canal crosses the summit here is 66½ feet lower than the surface of the old canal where it came across the summit. West of the bluff, on the State line facing west, is a plateau which extends north and south a distance of perhaps 15 miles and east and west a distance of 10 miles. The railroads coming from Lake Erie across the country to Pittsburgh have a 60-foot grade coming up from the lake. There is no road going south that has not grades of at least 60 feet to the mile. The Erie and Pittsburgh, which leaves the Lake Shore at Girard, has 62½ feet, and there is no summit level. Neither one of these roads has any summit level, but we have for our canal a summit level of 20 miles in length, and it is only a question of time in operation, whether or not it will be economical to make it more than this. A summit level 30 miles long can be made there if desired.

The question of the feasibility of the scheme as an engineering project we cannot now go into, because of lack of time. We will have to ask you to accept the statement made in the report, of which there is a copy in the library of the Society, that the construction of the canal is perfectly feasible. The principal question to be taken into consideration would be whether or not there is any commercial necessity for the canal. Here is the Mahoning Valley, where there are fifteen furnaces; in the Shenango

Valley there are twenty-three furnaces. In Allegheny County and the Mahoning and Shenango valleys 35 per cent. of all the pig metal produced in the United States is made. The "two valleys" made 1 250 000 tons in 1889, and Pittsburgh about 1 300 000 tons. The point in regard to the commercial necessity is this: if the railroads now carry material for this metal and deliver it to the furnaces along the valleys and at Pittsburgh, why should not they continue to do so? If the increase in the production of pig metal maintains itself at the rate of the last two or three years, in the year 1900 we shall be making between 17 000 000 and 18 000 000 tons of pig metal; and we must bring into this country more than twice as much iron ore as now. This cannot be done by the railroads. If the iron-making districts are to maintain their position the material must be brought to them. These districts have a right to an opportunity to keep up with the advance, but if not provided with additional means of handling their ores it is an impossibility for them to maintain their position. The canal would have a sufficient capacity to carry to these regions all the ore they want. A question that arises in every inquiring mind is, How is it possible that these railroads are so restricted in their ability to deliver freight?

A vessel bringing to a harbor an average cargo of ore carries perhaps 1 500 gross tons. Now, this quantity put into railroad cars will load about seventy-seven cars, occupying about 2 600 feet of track; while in the vessel, that ore occupies a space only about 215 feet long, and that is one of the essential facts to be considered in this matter. The railroads find it impossible to get to the furnaces with such enormously long trains of cars; one end is at the furnace, but the other is half a mile away. It is an impossibilty that the rate of supply to the furnaces by the railways shall be very much increased, because all available grounds are fully occupied by the railways. It is impossible to get a new railway into any one of several of the most important manufacturing towns. For these reasons the capacity of the railways is limited.

(Mr. Goodwin exhibited a map of the ship canal through certain towns, showing how fully the ground is taken up by railroads. This and other maps were donated to the library by him.)

These are the reasons on which we build up the statement that our canal is a commercial necessity. The cities along the lake shore do not appear to take any interest in the matter at all. Cleveland has an idea that the construction of a canal here would have an injurious effect

upon the trade of that port. Ashtabula is the largest ore-port on the lake. Fairport and Ashtabula are simply the lake ports of Pittsburgh and the two valleys; they have no business beyond vessels going to and coming from the ore-regions, so if the iron and coal industries were entirely removed, the business of these two ports would be wiped out. You are aware that Cleveland has an enormous manufacturing industry, as has also Erie. No city in the world turns out the number of stationary engines that Erie does. One concern alone turned out 1 200 stationary engines last year. The business of Cleveland, if the iron and coal industries were wiped out, would be reduced 70 per cent. While it might appear at the first glance to a Cleveland man that the construction of the canal might be injurious to that city, yet precisely the contrary is the fact.

The report of the Commission refers to a map that was printed by the Government about 1824. When this canal project was resurrected the War Department hunted up the plate and printed a lot of copies, of which the one here exhibited is one.

DISCUSSION.

E. P. North, Director Am. Soc. C. E.—Mr. Goodwin spoke about the old canal being abandoned. I would like to ask when it was abandoned, how long it carried freight successfully, the cause of its abandonment and the size of the canal?

Mr. Goodwin (showed on the map the route of the canal).—The canal was 28 feet wide on the bottom, 44 feet on the surface, and had a depth of 4 feet of water—on the old "Erie" prism. The summit cut of the old canal was about 8½ miles long. There was an average cutting of nearly 20 feet; maximum 27 feet. While the summit cutting was in progress the French Creek feeder was also in progress, and before the summit cut was completed the feeder was nearly completed, and a large amount of work done on other portions of the canal besides. The engineers in charge found it impossible to control the quicksand on the summit. There was a certain point to which they could reduce it, but beyond that they could not get it down. You can imagine the feelings of the engineer in charge, our Past-President, Milnor Roberts. Finally all the water available for the summit level and for 61 miles of the canal had to be pumped into the canal with a wheel and big bucket, such as they formerly used in Chicago. That was one reason why the canal was not a commercial success. It never had any depth in it because of the

quicksand. In 1863 or thereabouts the Pennsylvania Railroad, or those interested in it (or perhaps just as much the New York Central as the Pennsylvania Railroad, because the Central people furnished the money for connecting the project), began a railroad to parallel the canal. This road was completed in 1864, and is now operated by the Pennsylvania Company. As soon as the road was completed and in actual service, steps were taken to close the canal. Two bills passed the Pennsylvania Legislature on the same date; the first incorporating the Pennsylvania Company, the second authorizing the Erie Canal Company to sell their canal to any railroad company, in order that the railroad company might use the canal for the construction of railway tracks. There was a clause in the bill to the effect that, providing the railroad company did not use it for railroad purposes, they might use it for anything they wished. The old canal company having become much embarrassed (a large number of judgments having accumulated against it), Mr. Scott made a deal whereby he came into possession of the canal. There were other steps taken authorizing the selling of the canal. The canal was not immediately dilapidated; for several years after the purchase there was a show made of operating it; but it was not suitable for operating on account of the condition I speak of. The canal was then sold out by piecemeal. If it had been continuous to Pittsburgh, I think it would have been in existence still. Local business before that canal was built was nothing; the canal built up a coal export from the Chenango Valley. Until the canal was constructed there was no export of coal from that region. When the canal was built coal export sprung up; the canal made that trade possible and fostered it and was very useful to it; and the coal people would have liked to see the canal continued. When the railroad came in they were forced to give the railroad their trade.

Mr. North.—I desired, Mr. President, to bring out a direct statement in regard to my theory and perhaps the theory of many others—that the canals of this country have passed their usefulness in consequence of the increasing loads that engines can carry and the decreased cost of transportation on the railroads. If you build a canal that will accommodate a boat of 40 tons, you can then make no improvement in the economy of transportation on the canal. The only way in which you can improve it is by lengthening the locks so that you can put two or three boats through, or by destroying the locks and building larger ones. I hope the gentleman will not consider it personal when I say that he proposes to connect a navigation which is now of 16 feet depth and will be almost surely 20 feet within a few years with a 15-foot canal, and do business through it and with the largest iron center in this world. Now the boats already built on the lakes exceed in depth, width, length and capacity the locks which this commission is providing for them. As soon as the new Sault canal is built the size of the boats will be

increased; two or three years after the present lock, which has been called the new lock, at the "Soo," was built, only 11 per cent. of the vessels passing through that lock could have gone through the old locks. There will be a very decided change; they will begin to cheapen their rates on the lakes and necessitate the railroads decreasing their freight rates correspondingly. There is a tendency on the part of American engineers, when extending a waterway, to make it smaller than that which they connect with. This canal has 15 feet on the mitre sills.

In Germany an improvement was made in the Main below Frankfort, where there are 350 miles of navigation. They made locks on this improvement, with a depth of 7.3 feet, when the available depth in the Rhine is only 6 feet. If that course had been followed here he would have made his canal some 22 to 23 feet; that would have allowed the vessels passing through the St. Clair flats to maintain a fair rate of speed in the canal. The capacity of the vessel is said to vary with the cube of the depth of its channel. The cube of 14 feet is 2 744, and the cube of 20 is 8 000; that is to say, there is some three times greater capacity through increase of depth with corresponding increase of width. Wherever larger expenditures have been made for the improvement of waterways, the result has been an increase in traffic and decrease in the expense of conducting it.

In German waterways—and Germany could be put down in Texas and leave a wide fringe of land outside of it—the increased ton mileage on the waterways has been 66 per cent., and at the same time without any increase in the length of railroads the increase of ton mileage on them has been 52 per cent. One of the engineers of the German waterways says it is certainly without a parallel in history that a means of communication which has existed for centuries should double its traffic in ten years. The doubling of the traffic was made by improving the waterways, by increasing the depth of the locks, and increasing the navigable depth.

Mr. Goodwin.—In regard to the relative cost of transportation on the lake and by rail, the furnaces in Pittsburgh paid two cents a ton more on iron ore from Ashtabula to Pittsburgh (145 miles) than they paid on the same ore for a distance of 650 miles on the lakes.

If the report of the Canal Commission makes a statement it backs it up with figures. You can ascertain with very little trouble precisely the cost of doing any kind of work. The canal will be, if it is ever built by the United States Government, an absolutely free canal. A vessel of the "canal" class will carry between two and three thousand tons, and we have some that will carry more. The fact is, a vessel of that class can make a great deal more money by loading about a foot less than they do. They can make about 33 per cent. more money than under the present practice. In 1873 there was a craze on the lakes that was similar to the

narrow gauge craze, the idea that the only thing necessary was that they should build four or five barges and let a steamer tow the barges. I told them: "Let the barges go by themselves and the steamer go by itself; they would earn a great deal more money in the season than they could running together;" and that idea they hooted at.

As I said, this canal, if built, will be a free canal. Formerly the canal toll on grain from Buffalo to Albany was six cents per bushel; they reduced it to three cents; that was found to be too much. They saw it must be a free canal, or New York would lose its supremacy as a grain port. The people of the outside counties of the State of New York protested very seriously. Chautauqua County contended that it was not fair to tax a poor county like that $5 000 or $6 000 per year for the support of the Erie Canal. Kings County and the County of New York (being the City of New York) said to Chautauqua County: We are asked annually to pay $25 000 school money to Chautauqua County, and Chautauqua County is asked to pay $5 000 only for the support of the canal.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

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(Vol. XXV.—October, 1891.)

ON THE STRAITS OF JUAN DE FUCA, PUGET SOUND; AND GOVERNMENT IMPROVEMENTS ON THE PACIFIC COAST.

By BOLTON W. DE COURCY, M. Am. Soc. C. E.

WITH DISCUSSION.

In the year A. D. 1592 (according to the "Encyclopedia Britannica"), an old Greek pilot, in the service of Spain, discovered the straits of Juan de Fuca, so called after him. The "Chambers' Encyclopedia" makes a claim that it was visited by Captain Cook in the year 1778, but Kipps's "Narrative of the Voyages Round the World, Performed by Captain James Cook," shows that it was Nootka Sound, on the west coast of Vancouver's Island, that he visited. So we shall insist that "the old Greek pilot" has the honor of discovering one of the most wonderful inland seas on the surface of the globe, especially when its phenomena are taken into consideration.

With an average width of about 18 miles, from Cape Flattery to the Race Rocks, a little to the east of north from Port Angeles, the straits expand to the north to a width of 20 miles at Victoria, B. C., there dividing into several passages, the Haro Strait, the Middle Channel and Rosario Straits, to the south-east of which is Admiralty Inlet, the entrance to Puget Sound. The shores generally are bluffs, covered with a dense growth of red and yellow firs, with cedars and hemlock, and

are so bold that vessels can approach them to less than a cable's length, in almost every place, and the depth is wonderful—in many places exceeding 100 fathoms.

Spits are found of various sizes at nearly every headland where there is any change in the general alignment of the shore, causing a sudden alteration of the tide-flow. At Port Angeles is one of the most extensive; here the original bluff took a sudden bend to the south-east, and the resultant is a spit about 4 miles long and from ⅛ to ⅜ of a mile wide, with a shore so bluff on the harbor side, that the light-house tender can run alongside and make a landing with a gang-plank. On the other side of the spit the beach is more shelving. This spit encloses a most beautiful harbor, about 2 miles wide.

Fig. 1.

There is a total absence of any river or stream of navigable size, discharging into the straits; the Elwha, a mountain stream which has cut its way through a gorge, making a deep and narrow cañon, is the largest. There are, however, many beautiful streams flowing from the Olympic Mountains, furnishing the purest water and well stocked with speckled beauties.

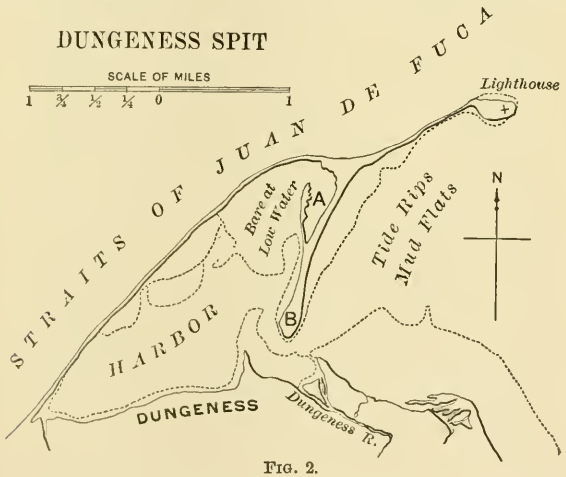


Fig. 2.

Twelve miles from Port Angeles spit are the curious spits of Dungeness. See Fig. 2. The Dungeness River rises in the Olympic range, and, as is seen in the present state of the harbor, brings from the foothills a great deal of silt. The interest here is concentrated on the question: In what succession were these spits formed?

The people cut a small channel through at the west end, expecting that it would create current enough to erode the mud flats and save the destruction of the harbor, but it had no duration, showing the same agents to be at work that first made the formation. Speculating, the author would suggest that the inner spit at *A* was caused by an eddy; while that at *B* was formed by the discharge of the river in combination with the ebb tide, the main portion being made by eddy. Southeast of this is Sequim Bay, at the head of which is another somewhat similar spit.

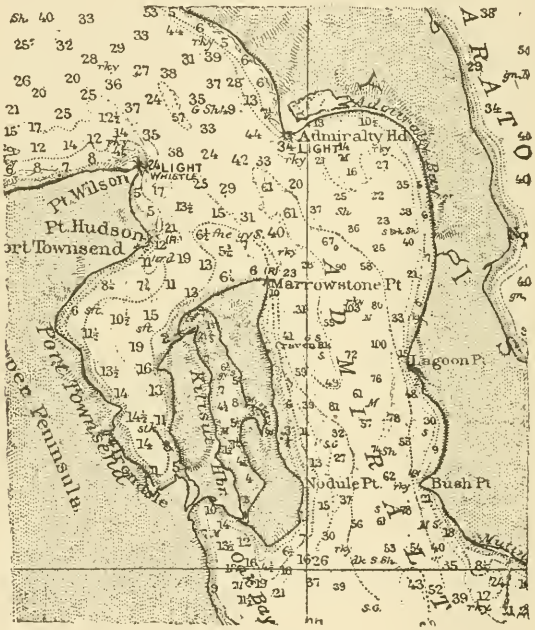


Fig. 3.

An examination of the chart, a small portion of which is given in Fig. 3, shows still further to the east a small bay at the south end of Port Townsend Bay or Harbor. At its south end, between the

waters of Oak Bay and Townsend, is what is called the Portage (from the circumstance of the Indians choosing this route to reach the upper sound, carrying their canoes over this narrow bar instead of the continuous water route around Marrowstone point). Through this small bay and bar, the people asked the Government to make an appropriation for a ship canal, claiming that it would shorten the distance to Seattle, Tacoma and other places on the upper sound, and enable the small stern-wheelers to reach Port Townsend in the winter, at which time they cannot be insured while rounding Marrowstone Point. This bar is evidently formed by the meeting of the tides in Oak Bay and Port Townsend, and if the canal were dredged the same agents would fill it again. The engineer to whom it was referred reported rightly against the scheme. East of Port Townsend Bay is Scow Bay, which also was once connected with Oak Bay and is yet, by a small channel which the littoral drift is continually working to close, and will, eventually, when the elongation of the channel reaches the Portage at the head of Oak Bay.

The "tide rips" are the most curious phenomena of the straits, Puget Sound and other waters of this system. A reference to a map will show that Puget Sound is formed of a number of long, narrow gulfs, connected with narrow passages lying principally in a northerly and southerly direction. The depth of these gulfs is wonderful, reaching 150 fathoms in places, and seldom, except at the heads, falling below 20. There are two tides daily, of different heights, and affording a "long and a short run out." The sound is 125 miles long and from 5 to 25 miles wide, with a coast line of over 800 miles in extent. The rise of the tide reaches from about an average of 10 feet at Port Townsend to 18 at Olympia, at the head of Budd's Inlet. All the water to supply this rise has to pass through Deception Pass, which is about 300 or 400 feet wide, and between Point Wilson and Admiralty Head, which are distant about 5 miles from each other.

In many places vast eddies and whirls will at times be seen on passing in a steamer, and while the water around the steamer is quite calm, and seems as though covered with oil, it will suddenly develop into a succession of short waves of wonderful height, and continue in this state for a time, when it will as suddenly calm down. These tide rips are caused by rocks and inequalities in the bottom over which the tide ebbs and flows with a rapid current. They occur often in great depths and are extremely dangerous to small boats.

Mr. Freeman's remark, in the discussion of his interesting paper on "Hydraulics of Fire Streams," is here in point. Referring to the effect of the small inequalities of the interior surface of the hose, he says: "As to the means by which such small roughnesses of surfaces are able to induce such powerful effects upon the friction loss, may we not consider that, instead of being due mainly to friction of the water upon the pipe, they are most likely due to the ability of these small projections to set the whole stream to its very center into a condition of turmoil and minute eddies, just as a single stone on the bottom of a smooth and deep canal is sometimes seen to produce a 'boiling' at the upper surface."

The shores of the straits and of Puget Sound are from 300 to 800 feet in height above the sea; and, as in the straits, Puget Sound has no rivers of importance or size flowing into it north and east of the City of Seattle. The beauty of the scenery of the sound requires more eloquence to describe than the writer possesses ; suffice it to say, the eye never tires when viewing it, and with the perpetual snow-covered extinct volcanoes, Mounts Baker and Rainier, sublimity as well as beauty is not wanting. Adjoining the sound are three lakes, Union, Washington and Samamish. A board of the Corps of Engineers of the U. S. A. is now devising a project for connecting these lakes with the sound by a ship canal. The difference of heights to be overcome, exclusive of the rise of tide, is only 12 feet, and the lakes being fresh water, will give the City of Seattle one of the finest fresh-water harbors in the world. The connection of all three will require a length of only 12 miles, and as the teredo is very destructive in the sound, Seattle will have an advantage over every other city thereon, as the fresh water acts as a cleanser of barnacles and all other *balani* that foul the bottoms of sea-going craft.

At Olympia, the State Capital, which city is situated at the head of Budd's Inlet, a project is prepared for the improvement of the harbor. Here, a stream called the Tum water enters the head of the inlet and has filled it up for quite a distance with silt. The project provides parallel bulkheads or training walls 400 feet apart, with a dredged channel 260 feet wide on the bottom; the excavation to be deposited or pumped back of the bulkheads, which are formed of two rows of piles, each with caps, stringers, breast-timbers, sheet-piling, and some fascines held in position with stone. The project calls for an appropriation of nearly $300 000. It is intended to furnish a channel 12 feet deep at lower low water, to be widened to 400 feet for 1 500 feet next to the city, and to keep the Tum water from flowing into it.

Strange to say, there is not one large, navigable river rising in the Olympic Range. The Soltuck, Colower and Beaver form the Quillayoute, which for some 10 miles from its mouth is navigable, but cannot be entered, from its bars; its embouchure is in the Pacific, some 40 miles south of Cape Flattery. The first harbor south of Flattery is Grays; it is called after Captain Gray, commander of the ship *Columbia*, who explored the river of that name (called so by him after his vessel), in the year 1792. Several small streams enter into this harbor, besides one of considerable size, the Chehalis; this is navigable by small steamers for some 25 miles from where it enters the harbor at Aberdeen, and for those that draw 15 feet at high tide, as far as Montesano, 15 miles. The United States Engineer in charge is now devising and estimating a project, to give 15 feet at all stages of the tide, as far as Montesano. The Chehalis River is formed by the Newaukum and several others, that rise in the foot-hills of Mount Rainier; as far as Montesano it flows through a hilly country; there it becomes tidal, flowing through flats covered for the most part with a growth of spruce, fir and other small timber. These flats have many sloughs taking off from the river, and in some cases entering again some distance off. There are several shoals, between Montesano and Aberdeen, which it is projected to remove; it is also proposed to dike the sloughs, in order to concentrate the water of the river. About a mile and a half from the mouth, the area is 25 300 square feet. The mean radius is 21.88 feet, and the wetted perimeter 1 156 feet.

Gray's Harbor (see Fig. 4) is about 15 miles east and west, of a somewhat triangular shape, and about 12 miles wide in the broadest part. It is a vast extent of mud flats, with a north, south, and middle channel, for the passage of the waters of the Chehalis and tributaries. At the mouth of the harbor, or rather 3 miles past, in the Pacific, is a bar of sand and breakers. The charts of the United States Coast Survey show a minimum depth at low water of $1\frac{3}{4}$ fathoms, when sounded by them. There is evidence that at other times, at low water, 22 feet depth has been found; the probability is that under the forces of nature, the bar crest varies, and at one time or another the water is worse or better for the passage of vessels, as the resultant depth is affected by storms and littoral drift, or the increased waters of the Chehalis in freshets. One of the duties of the United States Engineer in charge is to devise a project for this harbor improvement.

The sketch shows, that on the north shore, commencing at the mouth of the Chehalis River, is situated Aberdeen, a town of about 2 000 population; about a mile and a half west is Hoquiam, say 1 000 inhabitants, and about 2 or 3 miles farther west is Gray's Harbor, about 700 or 800. While all their interests are the same, the real estate agent has antagonized them, and each wants to be the metropolis of the region. On the south side are South Aberdeen, West Aberdeen, say 200 population; Drummond, a town in embryo, one or two others in the same condition as you travel west; then South Arbor, 25 population; Markam, same population, and lastly, on South Bay, Ocosta, a new town, with probably 700 or 800 population, and the terminus of the Northern Pacific

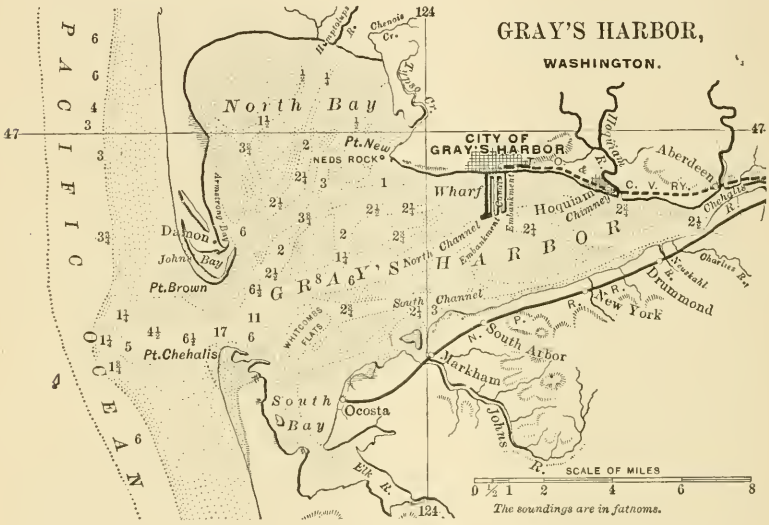


FIG. 4.

Railroad. Thus, the problem is to endeavor to satisfy all these interests; one calling for the improvement of the North Channel, the next for the middle, and the last for the South Channel to be made the main and permanent improvement. The channels are marked on the sketch. There are several shoals in these channels which heavily loaded seagoing vessels can only pass at certain states of the tide, while the main portions are deep enough for any vessels at all stages. There are vast quantities of lumber shipped, and the improvement of the channels, as well as at the bar, is called for and recommended; but the question is, what is the best project? It is proposed to endeavor to train the

principal part of the waters of the Chehalis into the north channel by spur dikes, so as not to close either of the other channels.

The shoals are marked on the plat, or sketch (reduced from the Coast Survey chart), by crosses, all the white spaces being mud flats, bare at low water. The tributary streams are on the north; the Hoquiam,

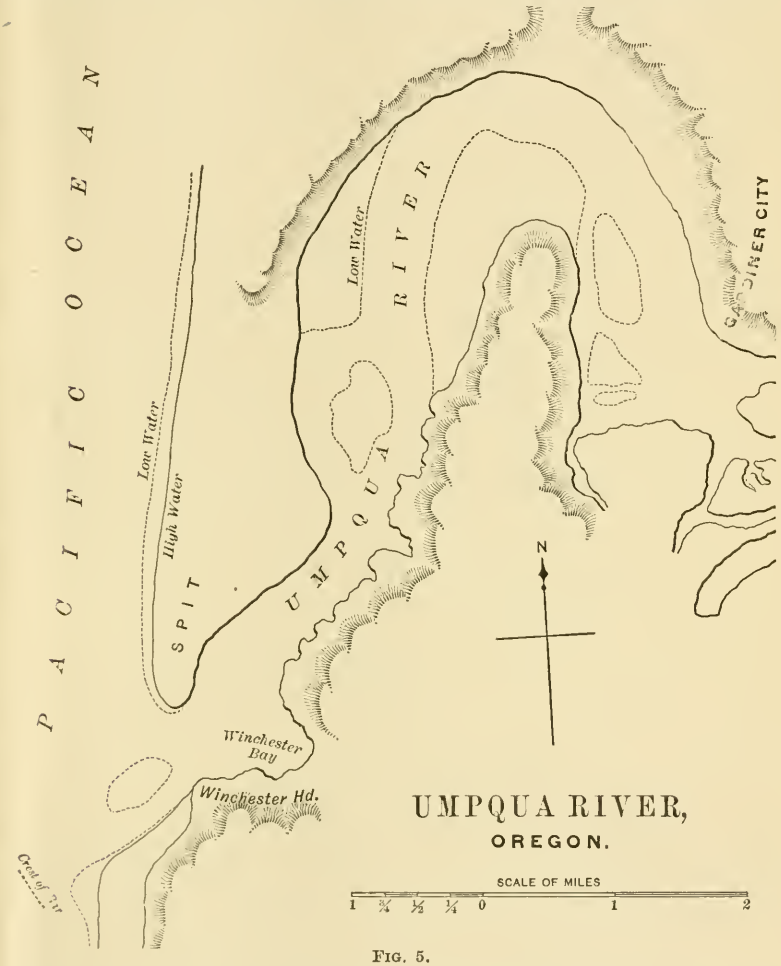


FIG. 5.

a fair stream, deep and navigable, being tidal for 10 or 15 miles, the Typso, Chenois and Humtuluips, with no channels over the mud flats; on the south the Charles, Neus Kah, Johns and Elk Rivers; the Johns being the only navigable stream, and that for 3 miles from its mouth.

Would not the best plan for improvement be to train the Chehalis through either the north or middle channel, so as to concentrate its entire waters, preferably the middle, leaving the flats on each side to be built over and improved in the future; also, to train the Hoquiam to a connection, acting the same way in the case of the Johns?

The next project on the coast is the improvement of Shoal Water Bay. This bay is about 20 miles south of Gray's Harbor; the Northern Pacific is making a terminus on the coast, also at South Bend. There are several tidal streams discharging into this bay; those navigable are the

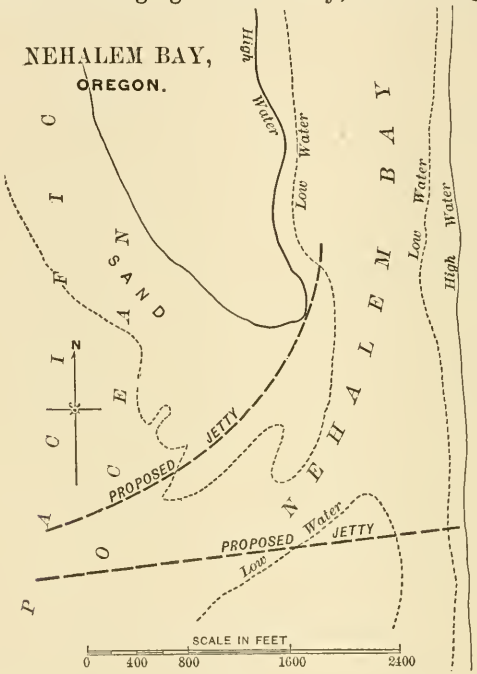


Fig. 6.

North, the Willapah and Nasel Rivers; these are all put forward for projects of improvement. The Northern Pacific is building from its station Chehalis to South Bend on Willapah Harbor. The Shoal Water Bay reaches to within about 5 miles of the Columbia River, at the mouth of which river Major Handbury, of the Corps of Engineers, has successfully built a jetty which has deepened the water on the bar, so as to enable the largest ocean vessels to enter the river with ease and safety. South of the Columbia the following rivers and bays have had either prospec-

tive projects, or projects approved, with regard to their improvement: The Nehalem, Tillamook, Yaquina, Siuslaw, Umpqua, Coos and Coquille.

The prevailing storm winds on this coast are the northwest and southwest. The waves are long and rolling, having a great distance of fetch, though this is affected to some extent by the bars at the entrances. The littoral drift seems to be in preponderance from the north. There are large masses of sand, in most cases, along the shore, which have a cycle of movement. Blown by the wind into the channel, they are carried by the freshets and ebb-tides out to sea, then by the littoral current or the storms are carried on to the banks to be blown again into the channel. It would seem as if an opening in the direction of either of the storm winds should be avoided, and on account of the long rolling waves of the Pacific a straight and parallel arrangement of jetties should be avoided, and an expanding embouchure from the gorge of the jetties for a short distance; but above all things, a diverging trace of the jetties as they enter the harbor, river or bay, as the case may be, except, perhaps, such rivers as have but a small tidal area.

The littoral drift should be made a special study. In several of the sketches herewith, the northern littoral drift can be easily seen; for instance, the Umpqua River shows it plainly at its mouth; Nehalem Bay also, and the Coquille, to a moderate extent.

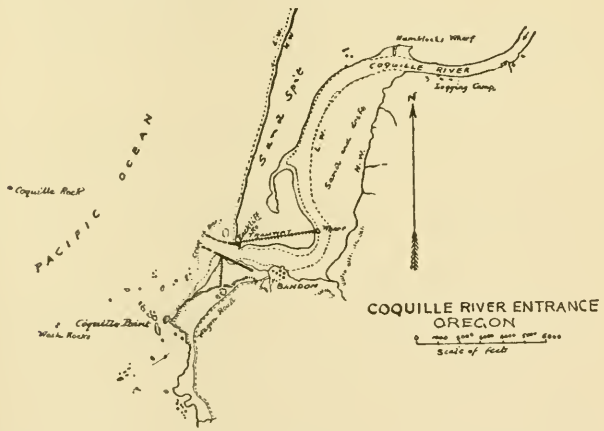


FIG. 7.

In the matter of the sands carried by the wind into the bays or rivers, it would appear that the proper remedy to apply is cultivation of such grasses and trees as can be made to grow under such circum-

stances. Rev. W. L. Rham, in "Flemish Husbandry," describes the improvement of poor white sand. Trenching and leveling is the first operation, then the sowing of broom. In three years, he states, the land can be used to raise buckwheat, and from that on, by the aid of manure, it becomes quite fair land for raising rye, potatoes and some other crops.

The "London Penny Magazine," March 30th, 1844, gives an interesting article on "Plantations on Drifting Sands." After describing the effects of these seaside shifting sands in Suffolk, Norfolk, parts of Ireland, etc., it describes, from the "Quarterly Journal of Agriculture," the successful process used there in the case of the sands of the outer Hebrides: "Square pieces of turf cut from solid sward are laid upon the drifting surface at stated intervals apart, being nearer together on steep places and farther apart in places of less declivity, etc. These turfs prevent the sand from drifting even in the intervals between them.

"Mr. McLeod, of the Island of Harris, adopting another method, has brought into permanent, useful pasture a number of acres of useless drifting sand. He cut the plants of the *arundo arenaria*, or bent grass, about 2 inches below the surface of the ground with a sharp spade; these are carried to the drifting sand and planted in a hole 8 to 10 inches deep; and a handful is put in each of these holes, which are about 1 foot apart. Neither wind, rain or frost destroys this grass; it is excellent for wintering cattle on, and in three years white or red clover will grow amongst it, provided it is well secured."

The pinaster or cluster-pine has been used with great success for fixing sands that drift. This was adopted in France in 1789 as a preventive of drift. In the Gulf of Gascony there are 300 square miles, which, before the attempt to fix the sands was made, could be only compared to a drifting, wavy, white sea. M. Bremontier began in 1789 the almost hopeless task of reclaiming this tract in the following manner: He caused seeds of the common broom to be sown in a direction at right angles to that of the wind, commencing at the seaside; protecting each zone with a line of hurdles, each zone also protecting the next. With the broom-seed he mixed the pinaster in the proportion of 4 to 5 pounds of broom to 1 to 2 of pinaster, immediately covering it with pine branches brought from the nearest woods. These were laid in a shingling manner, and in exposed places, small trunks of trees were overlaid. In six weeks to two months the broom-seeds had grown 6 inches; the

pinasters the first year grew about 3 inches, and in about seven or eight years overtopped the broom. In 1811, a commission appointed by the French Government reported 12 500 acres covered with thriving plantations. These plantations, called pegnadas, constitute the principle riches of the inhabitants, who are almost entirely supported by the preparation of tar and resin from the pinaster trees, raised in what were formerly useless, drifting sand dunes. Nearly the entire tract is now under cultivation.

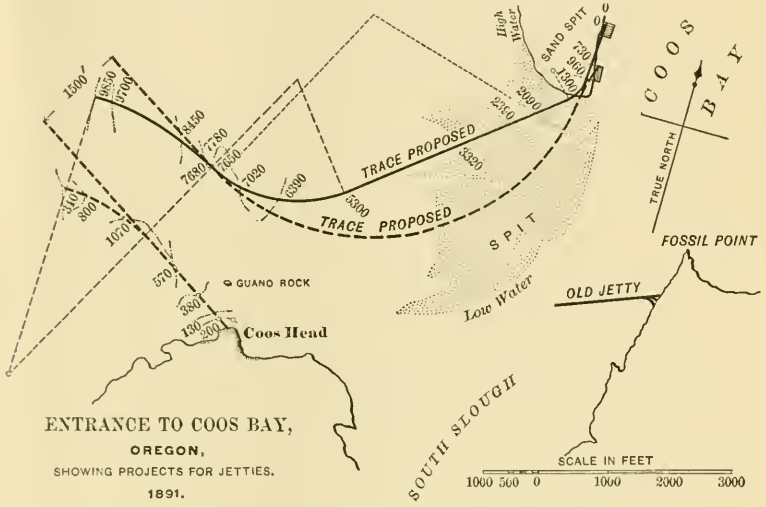


FIG. 8.

One of the most interesting bays on the Pacific coast is Coos Bay. A project for its improvement was approved in 1879, "to construct at an estimated cost of $600 000, a jetty of wood or stone, as may be found best, from a point 250 yards below the northern extremity of Fossil Point, on a line toward the east end of Coos Head, this line in plan curving, so as to be directed at its outer end to the head, or a little to the north of it." Since that time a board of officers of the Corps of Engineers have recommended a new project; they recommend "that the old jetty be not further extended, but that two jetties be built, one from Coos Head, and one from the southern end of the north spit, out toward the bar, ending at a distance apart of about 1 500 feet." The question is, What is the best trace for the jetties? The more southern or curved, convex-toward-the-river plan has been finally adopted. There is a large appropriation for this work. The work will be done

directly by the Government, experience proving that it is more economical than the contract system.

Yaquina Bay has another good appropriation. There are two jetties projected at the mouth of the bay. The project approved is, to

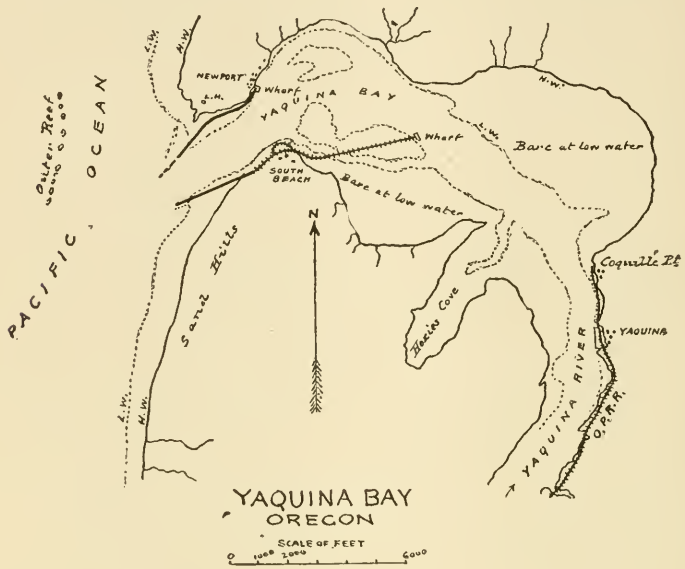


FIG. 9.

raise the south jetty to full high-water without extending it, and to construct a mid-tide jetty on the north side of the entrance, extending from the north head along and behind the reef, to a point opposite the end of the south jetty, and distant from it about 1 000 feet.

The Siuslaw River is the last project which will be referred to.

An examination of the preliminary sketch (Fig. 10) shows a peculiar course to the river, a large bend to the west along the south end of a sand bank which is washed out by the stream, following the concave side of the bend. There is, of course, a tendency to wash or erode into this sand to the north. Coming up from the south is a sand spit, with an incipient channel forming as shown. In devising a project, is it better to take the new channel and foster it with the system of jetties as shown, or take the river as now, confine it with the north jetty and afford a larger tidal area, that of the river not being of large amount? The estimates of the cost of construction do not differ enough to make the choice of

one of them be governed by the expense of the construction being a minimum.

The main object should be in all such improvements to obtain and maintain a good bar passage; and a study of these rivers and their char-

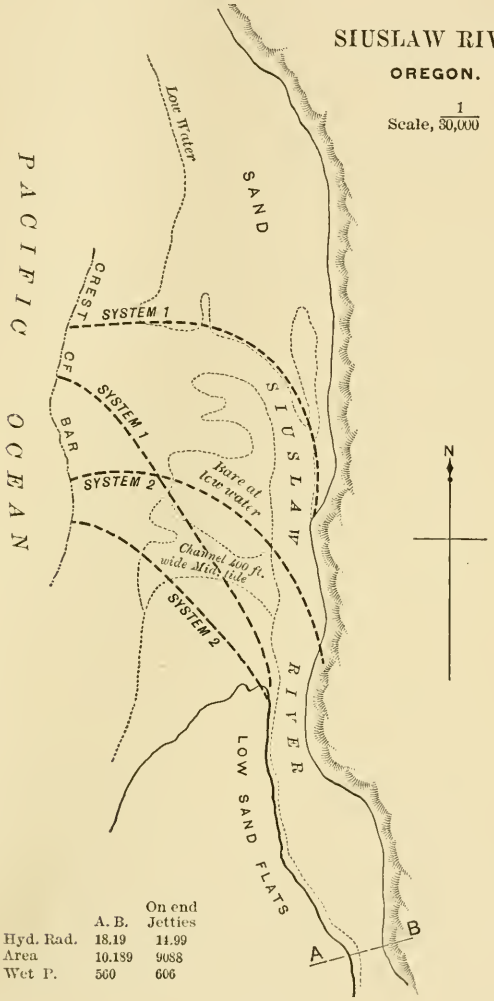


Fig. 10.

acteristics makes a person cautious in making up his mind. The hydraulic depth, mean radius, area, and wetted perimeter can be procured; but they will be very different in different places on the same

stream. The forms of the bottoms are peculiar, there being shoals, hollows and deposits of sand where they would not be expected. The engineer officers have, therefore, problems before them that are very difficult to solve, and more difficult also, after the course is decided as the best, is it to predict the result. The power of the Pacific waves with their long fetch is tremendous; and since the plan of appropriation is hampering, almost all projects having to lag at times on account of appropriations falling short, the works being in an unfinished state, are often injured by the storms and waves.

There are other projects in the matter of river improvement, which include blasting and removal of rock, snagging, etc. At the locks at the cascades of the Columbia some extremely fine work is being done by the Government. The officers of the Corps of Engineers on the Pacific coast are hard workers, and quite alive to the importance of the works under their direction.

The immense supply of the best timber, the late discoveries of coal and other minerals in the States of Oregon and Washington, are calling imperatively for the prosecution of these improvements with diligence and dispatch.

The author has approached this subject with diffidence and with the desire for increase of knowledge, from the discussion of the projects mentioned, by members of our society of matured knowledge and national reputation.

DISCUSSION.

George H. Mendell, M. Am. Soc. C. E.—Harbors and river mouths opening to the Pacific Ocean are entered by passing over outlying bars of sand, upon which the depth is less than it is on either side, seaward or landward. When the bar depth is small, the channel is subject to sudden changes caused by waves. The southwest storms develop waves which resolve into breakers in depths of 10 or 12 fathoms or more. Moderate gales cause disturbance of the bottom in smaller depths. Abrasion of the bed or erosion of adjoining shoals may cause sufficient movement of sand to nearly close channels, when not of considerable width and depth. Other influences produce similar effects. The summer winds blowing from the northwest transport large volumes of sand on adjacent shores. The currents—flood, ebb and littoral—are constant factors in the movement of sand to and fro. In the bights of

the coast a palpable littoral current makes to the northward during the summer months when northerly winds prevail. On some parts of the coast this movement is masked or reversed during the prevalence of storm winds from the southwest. Seaward of this littoral current, the ocean circulation due to the Japan stream, modified more or less by winds, follows the coast from north to south. Much remains to be learned of the extent and characteristics of these currents.

If we attempt to estimate the direction of the resultant littoral sand movement by the deposits shown on published charts, we shall be led to think that no general rule applies. A chart records conditions as found at a particular date. It affords no information as to either former or later conditions. The evidence of published charts needs to be supplemented by additional facts. It is maintained by some that the movement is in one direction in the summer and in the opposite direction in the winter, and in both cases obedient to the wind. This view seems probable. The evidence of the jetties constructed at the Columbia River, at Yaquina Bay and at the Coquille River, all on the south side, seems to demonstrate a sand movement of magnitude at these points to the north. At no point are north side jetties sufficiently advanced to afford evidence of the same character.

This interesting question has been studied, and it is hoped that the conditions of sand movement will soon be determined at a number of points, and the influence of the factors entering into this movement be severally ascertained.

Universality in occurrence of bars indicates that they are a necessary feature produced by the interaction of natural forces. The sand of which they are composed exists in abundance on the bed of the ocean and in adjoining shores. The bar in its position and depth represents at any period a relation of equilibrium between opposing forces—those that make to obstruct and those that make to open a passage. If the bar grows in extent or moves shoreward, or, keeping its position, lessens in depth, these are indications that the obstructing forces have gained upon their opponents. A reduction in the amount of the obstructing force, or a better application of the deepening force, brings about converse phenomena.

At the Columbia River the north head is Cape Disappointment, a basaltic cliff. The south head is Point Adams, the northern extremity of a stretch of sandy coast. It is retired to the eastward about 4 miles in rear of the Cape. From Point Adams a long and wide shoal known as Clatsop Spit makes out to the west, for more than 3 miles. This spit takes different forms in plan, and in its history has varied greatly in elevation, having been for an interval of two or three years several feet above low water. In this state it formed a natural jetty and served as far as its influence extended to concentrate the tidal ebb and flow in the main river. During other periods, its surface lay 6 to 8 or 10 feet

below the plane of low water, with one or more secondary channels through it of greater depths. In this condition the inflow and outflow were dispersed, a considerable fraction passing over or through the spit, thus reducing the amount of water circulating in the main channel. When the spit stood for 2 or 3 miles of its length above the plane of low water, the depth on the bar at low tide was 27 feet. In the other condition of the spit, the bar depth was 18 to 20 feet. These conditions of the bar and spit having been established by surveys and thus made matters of record, the proper method of bar improvement was thereby plainly suggested. If Clatsop Spit could be built up to stand several feet above the plane of low water and be maintained at this level, it was to be expected that increased depth on the bar would follow, and be maintained. Northerly movement of sand at this point is proved by the existence of the spit, and a barrier placed along the spit must arrest the sand carried by the flood tide or moved by the southerly waves. The barrier consists of a mound of riprap stone placed on mattresses of brush, all material being deposited from a timber tramway. This work is now advanced about 4 miles in incomplete condition, and stands for about 2 miles several feet above low water. Its effect has been to collect sand to the south of it, in such quantities as to create a large acreage visible at low water, in positions where the water was 6 to 10 feet deep four or five years ago; with the result of a marked increase of width and depth in the bar channel several miles seaward.

A similar construction on the south line of the entrance of Yaquina Bay has served to collect sand rising above low water to the south of the work, nearly to its extremity, and to increase the low water bar depth from 7 to 13 feet.

Fig. 9 in the paper shows the position of the jetties at Yaquina Bay. The axis of the channel prolonged passes to the southward of the outlying reef. A principle which guides in this project is to secure for the entrance, as far as practicable, protection from westerly swell, by placing it under the lee of the reef, in a position which shall permit free egress and ingress. Both jetties are yet uncompleted.

Jetties of this construction being formed of riprap stone of moderate sizes without bond, are not intended to withstand the blows of heavy waves. When placed upon a shoal where they serve to collect sand, they are protected by the deposit to windward. The waves break at a safe distance and so far from threatening the safety of the structure they give further security by increasing the mass of sand. Under such circumstances the work may be safely carried to half tide or higher. But in positions of real exposure to waves the work must be kept low, to permit the blow to pass clear; or the construction will need to be charged or strengthened by making the profile larger and by the use of heavier material. Thus far there has been no necessity of this kind.

At the Columbia River there is but a single jetty, and that is on the south side, the natural bank forming the other jetty.

While there appears to be no danger from the windward side, care must generally be taken to protect the work on the channel side. The ebb tide following the line of the work may undermine the foundation mattress which at the Columbia River is for part of the length between 12 and 18 feet below low water. The sea waves entering the channel, if permitted to race along the work, may produce the same effect, ending in the overthrow of a part of the structure and the opening of a new channel across the line. Generally there is danger to the jetty where deep water is found close along the line. A suitable alignment slightly convex to the channel affords some protection against both these dangers, which may be further obviated by a system of groins extended from the work channel-ward, which shall keep waves and currents at a safe distance and favor deposits between the groins. These jetties serve to increase the depth over the bar, but they do not efface it. It is moved out to deeper water and there is deeper water over it, but it remains. The water being deeper, the bar is less subject than before to change or deformation. The tidal currents flood and ebb having been concentrated in the same path, the ebb tide is discharged to the best advantage, and the outflow is enabled with more success to resist the efforts of the ocean waves, or other forces acting to obstruct; or, if not entirely successful for the time in preventing a certain injury, is better able, by persistent effort, to repair it after the storm has passed.

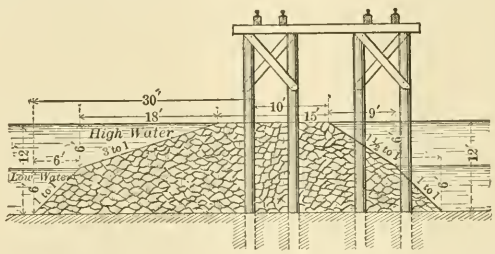
The first improvement undertaken by the Government on the Pacific Coast was that of a small lagoon on the Southern Coast of California, known as Wilmington Harbor. This improvement has proved to be notably successful. The tidal area of the lagoon is about 1 300 acres. principally made up of flats, bare at low water, and overflowed by the tide. The mean rise of tide is 4 feet, the maximum about 8 feet. The original low tide depth of water at the entrance varied from 1 to 2 feet. The existing depth of the same stage of tide is 13.5 feet. This depth is practically constant. Vessels drawing from 16 to 18 feet may now enter at high water. This improvement resulted from concentration of the tidal currents between jetties, made partly of timber and partly of stone, supplemented by a moderate amount of dredging. It is here alluded to as an illustration of the advantage of a debouche upon a coast protected from the frequent inrush of heavy sea waves. Storms on this part of the coast are rare and an outlying island in front of the harbor cuts off most part of the western swell. Upon a more exposed coast, no improvement of so small a harbor could have reached so successful a result. Harbors on more exposed parts of the coast with ten times the tidal area, and more than ten times the tidal prism, have in their present unimproved state much less water at their entrance than Wilmington has.

The works mentioned are now in course of construction, but none are completed. In every case there has been substantial improvement, but

it is yet too early to make up final judgment, which must be determined by time. A beginning has been made in the execution of a project for improving the entrance of Humboldt Bay, California, by two jetties. Plans and appropriations of money have been made also for Coos Bay, in Oregon, and San Diego, in California, but no work has been done.* Each of these three bays has a tidal area of 25 square miles or more, and a mean tidal expense of 80 000 cubic feet per second or more. Humboldt and Coos Bays debouch on a coast line exposed to heavy westerly swell and are situated in sand belts, where large quantities are in motion. Usually the prevailing low water depth on these bars is but 10 or 12 feet. In each of these cases one or both the headlands is sand. This is also true of every harbor entrance where improvement has been undertaken. This fact is mentioned as illustrating the prevalence of sand in a movable position, which in connection with the heavy waves that move it forms the great obstacle to permanent improvement.

Other bays of tidal prism sufficient to justify substantial works of improvement are Grays Harbor and Willapah or Shoalwater Bay, on the coast of Washington. The rivers of the Pacific Coast, except the Columbia, are too small to ever have considerable depths at entrance.

All jetties have perforce been built from a tramway, carrying a double car track. Mattresses of brush are built under the tracks, suspended from the caps, and when sufficiently loaded with stone are let down into place. At the Columbia River improvement a second line of mattresses



This Cut Shows Location of Tramways for Jetty Construction.

is placed on the channel side of the tramway. They are made as here, brought on cars and tilted into position. In the interval of time between driving the piles and placing the mattress, it is generally found that scouring occurs. This is an advantage in many cases, as it places the foundation of the work in a stable position, or at least in a position less exposed to undermining. The material of the jetty above the mattress is always the stone of the country, discharged from tilting cars. This is enforced by considerations of economy. The best work is done by stones of varied sizes, the smaller filling the voids of the larger, and together forming as close work as can be expected from the

* The north jetty at Coos Bay has been begun and extended in a partially completed condition for a distance of 4 500 feet.

enforced methods of construction, and sufficiently good for the purpose in view. The maximum weight at the Columbia River improvement is about 6 or 7 tons.

At the Coquille River the channel originally made to the south, following a dangerous course through the reef. The effect of the south jetty has been to make a direct course to the west free from hidden dangers. The original depth was 3 feet. It is now 6 feet. The project aims to secure 8 feet, to which the rise of the tide will add 5 or 6 feet. The position of the jetties as projected is shown in Fig. 7. They are as yet incomplete.

Wilmington is a tidal harbor, there being no fresh water drainage worth mention. San Diego is also a tidal harbor. The harbors on the northern coast are mainly tidal, although they receive during the rainy season a certain amount of drainage. Even in the Columbia, which is one of the large rivers of the Continent, navigation at its mouth is mainly dependent upon the tide in the sense that if there was no tide the entrance would not admit large vessels at all seasons. The same is true in even greater degree of San Francisco Harbor, in which backwater is quite insignificant compared with the prism made by 400 square miles of area covered by the tide twice per day.

The mean rise of the tides increases with the latitude; being about 4 feet on the southern coast of California, and about 7 feet at the Columbia River. Exposure also increases with latitude.

Restraint of sands which may be moved in considerable quantities by natural agencies to new positions within the tidal area, is a necessary feature in a project for harbor improvement. The tidal area may be lessened by encroachment, thereby producing a reduction of prism, and the swerving force may be further diminished by the duty imposed upon it of carrying foreign matter, leaving less force useful for erosion where needed. Moreover, the sand carried by the ebb may be deposited in the sea in unfavorable positions. The project for improvement of the entrance at Coos Bay, which is perhaps the most conspicuous instance we have of wind movement by sands, provides for their restraint by plantations. The sands here overwhelm houses and fences, and the tops of trees appearing through the drifts and other indications justify the inference that the accumulations are a hundred feet or more in depth. These sands reach tidal waters in very large but unknown quantities, and together with the additional amounts cut from the beach by waves or carried by currents, this element becomes a serious subject of consideration, for the reason that it is a great obstacle in the path of improvement. A beginning in planting has been made. The grass variously designated as *Arundo arenaria*, as Holland grass and as Helm, used with great success in Holland and more recently with very favorable results in San Francisco, has been set out to some extent. Other grasses, both indigenous and exotic, shrubs and pines, are undergoing

trial. There is always the question to be solved by experiment whether an exotic plant can thrive under foreign circumstances of climate and exposure. Holland grass grows well in the sands of San Francisco peninsula, and transplanted to Coos Bay a few months ago, it gives promise of life, but it is yet too early to be certain of its health and fertility.

B. W. De Courcy, M. Am. Soc. C. E.—Since writing my paper I have been engaged as Chief Engineer of the survey of Grays Harbor, for the Harbor Line Commission of the State of Washington, and the more I look at this harbor the more I am impressed that it affords the most interesting problem on the Pacific coast in Washington. On examination of the chart it will be found that the actual channel shows no soundings taken in it, the line of soundings on the north being north of the channel, that to the south being south of it, and the original chart as first drawn shows the 12-foot curve on the north side, curving and connected to the north; the south 12-foot curve being *vice versa*. When they changed it, or why, is not known, for vessels drawing as much as 17 feet have been passing out timber laden for many years, and there has never been but one accident on this bar, and that happened through mismanagement. The United States Coast and Geological Survey has made a survey which will give the correct depth in a short time.

There are no teredos to be found anywhere in the harbor. When the tide is running out the water has been found perfectly fresh, even on the bar, which is three miles outside. I think that this river, or rather so-called harbor, can be considered as an estuary, like the mouth of the Thames or Humber. Old sea captains who have been sailing in and out for over 20 years have assured me that they have never found under 19 feet on the bar, and that they have come in at all stages of the tide.

I had a tide gauge at Ocosta, and in order to ascertain how much higher the water was on the north end of the south spit, I had a line of levels carefully run from opposite to that place. I found at all times the water a foot higher there; in the center of the channel it must give a greater excess. At Aberdeen the water in the river (1 600 feet wide, 24 feet deep, at low water) is never still. The current instantly changes. I have seen logs coming in with the tide suddenly stop and immediately pass out again; and I have seen the tide running in strongly and the water falling eight-tenths on the gauge, indicating a subcurrent. The Chehalis River is deep and discharges much water. The lumber traffic is enormous, the supply inexhaustible; it must be seen to be appreciated. Some quarter sections will furnish as much as 10 000 000 feet, board measure, of merchantable lumber. The improvement of the harbor, where the railroad reaches within 5 miles of the ocean, is imperatively demanded. Ocosta has a small, though deep and well protected harbor, with a deep channel to the bar.

If my paper has done nothing else but draw out the interesting discussion by Colonel Mendell, I think it successful. Certainly Colonel Mendell knows the correct position of the reef at Yaquina, and whatever he says I should concur in. My information came from a gentleman named McMillan, a civil engineer, who was in Government employ, and I quote his words: "The reef at Yaquina lies in front of the jetties, and if they are extended too far would run up against the reef. Vessels enter the bay from the south end of the reef."

The State of Washington has made a fund for the improvement of her own harbors. The State appropriates 75 per cent. of the proceeds of the sale of the tide lands for improvement of harbors, to be laid out under the direction of the Harbor Line Commissioners. The next thing the State requires is an engineer, who should be the most skillful man to be had in all things relating to harbors.

The South Spit, at the entrance to Grays Harbor, is a black sand which contains considerable float gold. It was worked by Chinese once, but abandoned for want of fresh water to work it. They are said to have secured at the rate of $1.50 per day to the man, even with salt water.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXV.—October, 1891.

SOME RECENT EXPERIMENTS WITH DYNAMITE ON AN OCEAN BAR.

By O. M. CARTER, M. Am. Soc. C. E.

WITH DISCUSSION.

During the months of July and August, 1891, some experiments were made by a committee of the Board of Trade of Brunswick, Ga., with a view to temporarily increasing the depth of water over the ocean bar at the entrance to that harbor. The inner harbor, with depths in 1880 of only 9 feet at mean low water, has been improved under the direction of the United States Engineer Department until there now exists a minimum low-water channel depth of 15 feet. The outer harbor, which comprises the anchorages in Brunswick River and St. Simon's Sound, has ample depths for all classes of vessels. The entrance to the harbor is, however, obstructed by the ocean bar, which extends in a horse-shoe shape from St. Simon's Island on the north to Jekyl Island on the south. The bar has not changed much in location or in general direction within the last thirty-five years or since the date of the earliest authentic survey, but the channel across it has shoaled during that period. The mean low-water channel depth, which, in 1856, was 15 feet, was found by the survey of 1890-91 to be, in places, not more than 13.2 feet. The mean rise and fall of tide is 6.8 feet. The outer 18-foot contour had not moved appreciably seaward during

the period named, but the navigable bar channel had shifted its position toward the south, the 6-foot contour of the "north breakers" having moved southward about one-half of a mile. Some slight shoaling had also taken place in the deep pocket inside of the crest of the bar. The general outline of the locality is shown in Plate XC. The bar, which is 4 miles from land, is a narrow one, the outer and inner 15-foot mean low-water contours being separated by a distance of from 650 to 750 feet. The crest of the bar to a depth of about 20 inches, is composed of rather coarse sand and small particles of broken shells overlying sticky blue mud, which contains occasional shell fragments. The bottom in the deep pockets, both on the sea and on the harbor side of the bar, is composed of blue mud.

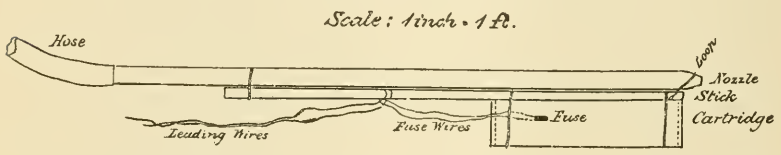
Some two years since, some temporary deepening of this bar was accomplished by dragging across its crest during the ebb tide a special form of scraper or harrow devised by the writer for use elsewhere on the coast, but the sand was difficult to put in suspension, and it was found impossible by that method to secure more than a few inches increase in depth.

The commercial interests of the port being seriously affected by the condition of the bar, it was determined early in the present year to renew the attempt to obtain deeper water. In June, a small sum of money was appropriated by the City of Brunswick to be disbursed under the direction of a committee of the Board of Trade, and the writer was consulted as to the best method of expenditure. The methods recommended, viz.: dredging, or sluicing by propeller or water jet, could not be carried into execution on account of the impossibility of procuring a suitable plant at the proper time. The committee thereupon determined to try high explosives, notwithstanding the fact that somewhat similar work at the mouth of the Mississippi and elsewhere had proved unsuccessful. The material used comprised one tugboat, used as a tender, and for shifting anchors, etc.; one schooner, about 15 tons burden, dismantled and floored over to make a working platform; one bateau; one small skiff; one diving outfit, complete; one Gould force pump, No. 12, with 3-inch suction and 2-inch discharge, requiring four men at the brakes; six anchors; six coils of 3-inch rope; 6 000 pounds dynamite No. 2, in cartridges; one Laflin & Rand exploder, fuses, lead wires, etc.

The working party, as finally organized, consisted of one superin-

tendent, a member of the Board of Trade Committee in charge of the work, one overseer, one master of schooner, one diver, one assistant to diver, eight sailors and laborers. In addition, the crew of the tugboat rendered assistance when necessary.

The ranges for giving direction to the work having been established, the working boat was moored in position by means of bow, stern and breast lines, all of long scope, to permit of considerable freedom of movement without shifting anchors. The boat was always kept with the stem to the current. A stop was placed on the bow-line to mark the point to which the boat must return after being hauled out of the way of an explosion. The stop was moved about 15 feet each time, thus spacing the charges at that interval. From 10 to 20 pounds of dynamite were used in each charge. It was enclosed in a tin cylinder about 20 inches long and 5 inches in diameter, closed at the ends with wooden disks. The fuse wires were brought out between the wood and the tin. The cartridge was prepared by lashing it to a stick 4 or 5 feet long, to which the fuse wires were fastened to take up any accidental strain. The stick was in turn lashed to the nozzle of the hose leading from the force pump, the nozzle projecting about 2 inches below the cartridge. The whole arrangement is shown in the following sketch:



The joints between the fuse wires and the lead wires were not insulated, but care was taken to prevent the bare portions from coming in contact. When all was ready, the diver took the cartridge and was lowered, the pump was started, and by means of the water-jet the cartridge was sunk in the bottom to a depth of from 2 to 3 feet. The upper lashing of the nozzle was then cut and the hose drawn on deck. The diver followed and the boat was hauled about 100 feet away, the lead wires being paid out. The charge was exploded and the boat brought back to its next position, 15 feet from the last. By experiment, it was found that 15-pound charges spaced 15 feet apart gave the best results. In placing a charge, the diver was under water about two minutes; the pump was worked about one minute; and the charge was exploded in

about four minutes after the diver went down. The interval between consecutive explosions was from eight to ten minutes, but as the party was not well organized and as much time was lost in going to and returning from the work, in shifting anchors, in adjusting ranges, etc., the explosion of twenty cartridges was considered a good day's work, the greatest number exploded in any one day being twenty-nine. Each discharge produced a crater about 10 feet in diameter and 5 or 6 feet deep. The material was loosened, however, a foot or more deeper, and the walls between craters were more or less broken down. The diver states that after the first few charges the bottom became so shaken and loosened, that succeeding cartridges sank into the sand more readily than did the first ones. Work was begun on the line *C D*, Plate XCI, at the inner edge of the bar, and an attempt was made to carry through a channel 50 feet in width. Considerable difficulty was experienced in keeping on range, and in the beginning the cut in places was nearly 400 feet wide. The work, however, progressed in a satisfactory manner for a distance of about 250 feet, when a compact deposit of sand and shell was encountered, through which but little progress could be made. The direction was then changed toward the north, branching off from the original cut near its inner end and crossing the outer edge of the bar about 500 feet to the northward of the original line of work. It is probable, however, that its position was not well defined or adhered to, inasmuch as few traces of it are now in existence. Operations were suspended about August 20th, after a vessel drawing 20 feet 9 inches had gone to sea through the new channel.*

In accordance with the request of the committee having charge of the work, a careful survey of the bar channels was made under the direction of the writer on September 7th, 1891. A study of this survey shows no appreciable gain in depth as a result of the work done, the mimimum low-water depth being now 13.3 feet, as against 13.2 feet before work was begun. In the old channel there is now a depth of 13.3 feet, as against 13 feet shown by the survey of 1890–91. It is probable that much more material was moved during the progress of the work than is shown by the survey, which gives only the final results obtained.

The cost of the work was about $6 000. The result was an excava-

* The vessel went out on the high water of spring tides, and could, doubtless, have gone out through the old channel, the survey of September 7th showing the same depth in the two channels.

tion of about 6 500 cubic yards from the harbor side of the bar and a fill of about 5 000 cubic yards on its seaward side.

Referring to Plates XCI and XCII, it will be seen that a pocket 450 feet in width, 250 feet in length and with an average depth of about 1.5 feet has been cut into the inner edge of the bar, but that the crest of the bar has been moved seaward and that more or less shoaling has taken place on its outer slope. The vertical fill is, in some places, as much as 4.3 feet, and averages one foot in depth over an area of about 12 000 square feet. The width of the bar between the 15-foot mean low-water contours along the line *C D* has been reduced from 750 feet to 550 feet, and little shoaling has taken place on the seaward slope. There being along that line deep water immediately in front of the crest of the bar, it is possible that had the work been continued as originally proposed, a 15-foot channel across the bar could have been obtained.

While it is possible by the use of high explosives judiciously employed to obtain an increase of depth on ocean bars composed largely of mud or other material easily moved and of low enough specific gravity to be carried in suspension, there is nothing in the experiments recently made to indicate that the desired results can be obtained in that manner as surely or as economically as by other well known methods, especially on sites of great exposure where the work is frequently interrupted for days at a time by rough weather, and where the material composing the bar is such as to be with difficulty carried away by the tidal currents.

DISCUSSION.

Charles B. Brush, M. Am. Soc. C. E., Director Am. Soc. C. E.—Several years ago when some parties attempted to remove a gravel bar in the harbor in New York, not by explosives but by a system of water jets, the same unfortunate results were obtained. They succeeded in stirring up the material and loosening it to a considerable extent, but there was not a sufficient tide to carry the material away. It simply dropped back into its place, and after an expenditure of $6 000, it was found that the bar had not been removed to any appreciable extent.

F. Collingwood, Secretary Am. Soc. C. E.—The remark made by Mr. Brush reminds me of the visit I made in the summer to see the dredging that was going on in the lower bay here. It was very interesting, and the process has been entirely successful. It was done by a large centrifugal pump and a kind of plow attached to the suction pipes, which are some 15 inches in diameter and are flexible. The discharge

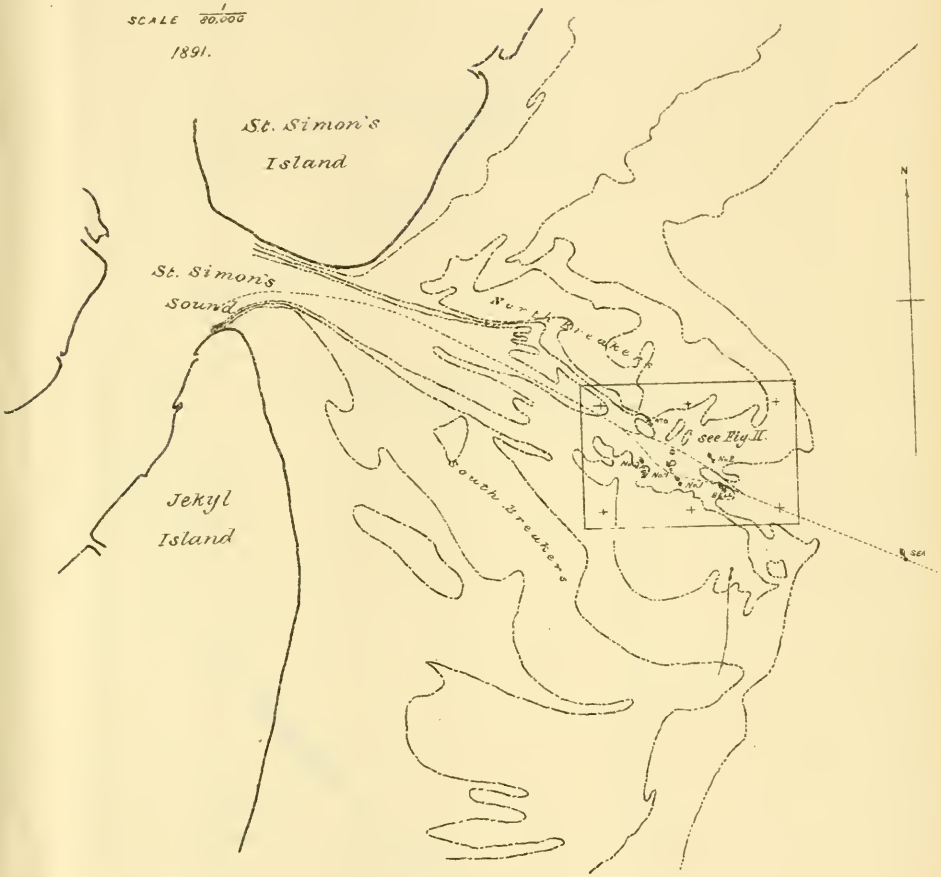
PLATE XC.
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXV. Nº 506.
CARTER ON EXPERIMENTS
WITH DYNAMITE ON AN OCEAN BAR.

BRUNSWICK, GA.

GENERAL SKETCH OF OUTER BAR

SCALE 1/80,000

1891.



Legend:

6 ft. m.l.w. contour shown thus : ——·——·——
12 . . . . . ——————
18 . . . . . ——————
Sailing ranges . . ——·——·——

PLATE XCI.
TRANS. AM. SOC. CI. / ENGRS.
VOL. XX / No. 406
CARTER ON EXPERIMENTS
WITH DYNAMITE ON AN OCEAN BAR.

BRUNSWICK, GA.

DETAILED SKETCH OF BAR

SCALE $\frac{1}{9600}$

1891.

Legend

15 ft. m.l.w. contour before work was done : ———— , after : +—+—+—+—+

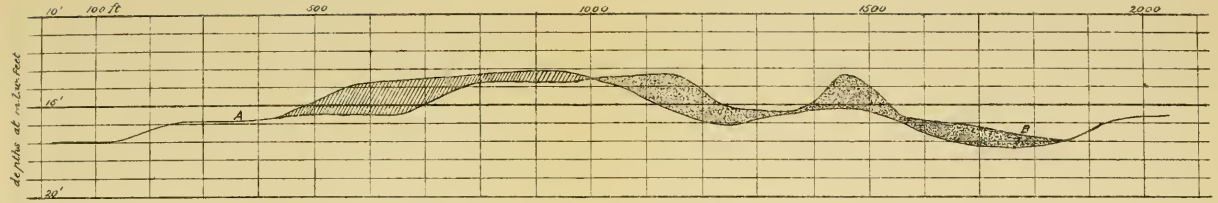
13 " " " " " " : - - - - - - - - , " : —·—·—·—·—

12

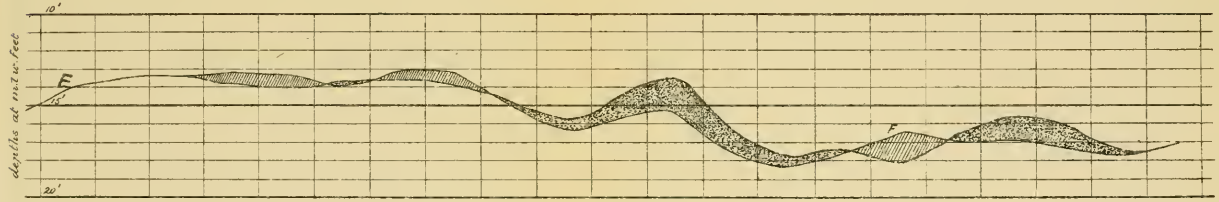
PLATE XCII
TRANS. AM SOC. CIV ENGRS
VOL XXV Nº 506
CARTER ON EXPERIMENTS
WITH DYNAMITE ON AN OCEAN BAR

BRUNSWICK, GA.

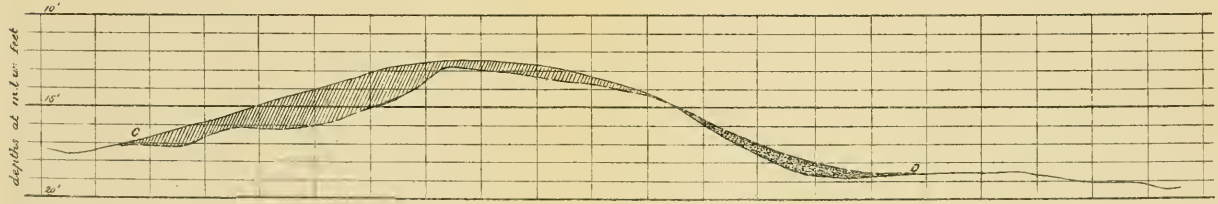
PROFILES SHOWING CHANGES IN BAR.



Profile on line A-B, new channel.



Profile on line E-F, old channel.



Profile on line C-D, proposed channel.

Surface of bar before work was done in full lines, after in broken lines.

Cut shown thus: ▨, Fill shown thus: ▨

is from 8 000 to 12 000 gallons a minute, and the result is that it brings up everything in the way. There is a hand hole with a cover in the top of the plow, which when opened allows the removal of any obstruction. The plow is very heavy, and it is sometimes weighted to make it take hold of the bottom; it is about 4 feet long and has partitions across it to break up any clay masses that may be met. The pumps were designed by Mr. Edwards and are some 3 feet in diameter. They have three blades only and are so arranged as to pass masses nearly a foot in diameter. At the outer end of each blade is a wrought iron plate, which if anything enters which would rupture the blade simply bends and is thrown out with the water.

The great velocity of the water brought up a heavy load of mud, sand, gravel, bricks, or anything else that the flow loosened. This was thrown into hoppers and settled to the bottom, while the water flowed off over the top. In about forty minutes 3 400 yards were lifted and deposited in the pockets of the dredge. The soundings completed over the outer channel in June show that in two years there has been very little shoaling and the channel seems to be permanent. This work was carried on always through the winter and almost without cessation.

Mr. A. Fteley, Vice-President Am. Soc. C. E., stated that in Mr. Eads' dredging at the St. Louis Bridge, a piece of cast-iron 10 inches long and 4 x 5 inches wide was sucked up and passed through one of those pumps; so they will carry up almost anything.

Mr. Brush.—The plow in the case I mention was a series of water jets that were drawn along the top of the bar, a large quantity of water being forced through these jets under very great pressure. It was hood-shaped and something in the nature of a rake, and was intended simply to stir up the material. The current was relied upon to do the rest. The quantity of material disturbed was large, but it seemed to settle down in its old place. The tide did not carry it away. In relation to this question of tides, I would like to call your attention to the fact that practically there are always three tides over that bar just as there are in the Hudson River. When the tide is flowing out on the surface another current is usually flowing in below, and still further down another is flowing out. I have frequently noticed that when material in suspension in the water is moved in one direction on the surface, it is forced back by the lower current.

J. F. Lewis, Assoc. Am. Soc. C. E.—Referring to the paper by Captain Carter, I think much better results would have been obtained by putting the charges deeper, say 6 feet or more. The same amount of powder would then have broken a crater at least 12 feet in diameter. If this could not have been done with the force pump and nozzle, it could have been accomplished with a small pile driver worked with an engine or by hand, putting down three or four holes across the line of excavation, charging each hole so soon as the bar or drill was withdrawn,

connecting the charges with wire and firing them all at the same time with an electric battery. This would have opened up a clean cut, leaving no walls between the holes, and I think the rush of water into so large an excavation would have washed out more of the material. This plan I think would have given them an excavation that could have been carried back on the proposed line without the trouble that "was experienced in keeping on the range."

Milnor P. Paret, M. Am. Soc. C. E.—There are several points brought forward in Captain Carter's interesting paper which cannot but be of general interest, and are particularly so to the writer of this article, since he was employed on the survey of 1890–91, referred to.

Captain Carter has described in detail the method of procedure, and shows us by a comparison of surveys made before and after the work was done, what the results were. However, what would appear to be the most interesting and instructive lessons to be taught by such experiments, have not been touched upon. I refer to the directions and velocities of currents at the time of the experiments, in connection with their ability to transport the material removed. That such points as these are not mentioned is due no doubt to the probability that no record of velocities, directions, etc., was kept by the parties on the work during the period of experiments.

The profiles indicate that all the work was done during the ebb tide, and, it may be supposed, on the strength of the tide (lasting three or four hours), "The time between consecutive explosions was from eight to ten minutes, and twenty explosions an average or good day's work." Twenty explosions, ten minutes apart, would occupy three and one-third hours.

Records of range of tide, times of high and low water and times of slack water, velocities and directions of currents, etc., would have been valuable in discussing a possibly proposed plan of work for permanent improvement in connection with the velocities to be generated and maintained for a permanent channel across the bar. It would seem possible that the reason so little apparent fill is shown on the seaward side of profile "C. D." is that the direction of the current might have carried the material toward "E. F.," where the fill appears excessive.

Soundings taken on an ocean bar, 4 or 5 miles from shore, and reduced to the then low water by readings on a gauge still farther from the locations of soundings and where the tidal conditions are somewhat different, can hardly be relied upon to give results within half a foot either way of the proper depth. The force and direction of the wind, the height of the sea or swell running at the time, the reliability of the leadsman and of the tests of the lead line, and the accuracy with which the soundings are plotted, are all sources of error, and even when all are reduced to their lowest limit, a margin of half a foot either way would seem to be but a small probable error. This fact would appear

to be illustrated on profile "E. F.," where at "F." there is shown a cut of 1½ feet, in 17 feet of water; this point "F." is about 650 feet from the apparent termination of the blasting on the line "E. F." It would hardly seem probable that the experimenters would move such a distance and there work in nearly 17 feet of water.

The result of the experiments has justified Captain Carter's opinion and conservativeness in not assuming the responsibility of recommending the expenditure of $6 000 on experiments promising such questionable results; but we cannot agree with him in recommending sluicing by propeller or water jet, in the face of the fact that a somewhat similar experiment made with a harrow, some two years previous, at the same point, proved that as the sand was difficult to put into suspension, only a few inches increase in depth could be obtained.

Bolton W. De Courcy, M. Am. Soc. C. E.—It appears to me that attempts to remove or improve a bar, without some plan to control the forces producing it, will only be of temporary and therefore questionable benefit; however, the immediate results seem to encourage an attempt on a larger scale, that is, a system of simultaneous blasts over the entire crest. The material composing the bar, its gravity and the velocity of the current must be carefully considered for a successful result.

Lewis McHaupt, M. Am. Soc. C. E.—In reading over the interesting description of these experiments upon the bar at Brunswick, Ga., described as having been conducted by "one superintendent, a member of the Board of Trade Committee in charge of the work," the first query suggested is, where did this civil body obtain the authority to conduct these operations in a navigable channel within the jurisdiction of the United States, and how does it happen that this report is published in advance of the annual official report of the Chief of Engineers?

It would seem from the context that the funds were provided by the City of Brunswick, "to be disbursed under the direction of a committee of the Board of Trade," and that the resident Government engineer "was consulted as to the best method of expenditure"; that he "recommended dredging or sluicing by propeller or water jet," which could not be carried into execution, and that "the committee thereupon determined to try high explosives," etc. Thus the responsibility for the method, the provision of the means for its execution and the conduct of the experiments are all ascribed to the Committee of the Board of Trade, and the function of the Government, as represented by its official, appears merely to have been to report the results unofficially through this society for the benefit of the profession.

These results might have been clearly anticipated, as the attempt to create a deeper channel on the outer bar without recourse to any modification of physical conditions which would permanently, or even temporarily, change the regimen of the currents, could not be expected to produce more than a temporary change—and as the sequel shows, the

modus operandi was defective in this, that the cut was begun on the inner slope of the bar instead of on the outer. The material loosened was therefore carried to the outer slope by the ebb and dropped in the path of the proposed extension, thus tending to increase the cost had the work been carried across the bar. It appears from the survey of September that no trace of the cut remained; the mere increase of one-tenth of a foot is altogether too small to be significant, as it is within the error of an ordinary sounding observation on soft bottom or an undulating surface, such as is almost invariably found over bars.

The fact of a vessel drawing 20 feet 9 inches having gone to sea through the new channel does not seem to have any relation to the matter; for if the original depth was 13.2 and the mean rise 6.8 feet, it would make 20 feet, while a spring tide would readily give a depth over so short a bar sufficient to float a vessel of this draught without having improved the channel. The expenditure of $6 000 to secure a net gain of about 1 000 yards with no prospect of permanent improvement is an experience which gives no promise of success, except in cases where the bottom has not been formed by gradual accretions due to natural agencies, but where rock or hard pan is found *in situ* and it is beyond the power of the current to remove it.

J. F. Le Baron, M. Am. Soc. C. E.—In the case described in the paper under discussion, it is probable that better results might have been obtained if the charges had been sunk deeper in the sand, which could have been done by using an iron pipe with the water jet pipe passed down inside. But after all we are tempted to exclaim, *Cui bono?* Supposing a continuous depth of 30 feet at low water had been obtained across the bar, how long could we expect it to keep open without protection or training walls? The result of this experiment was, that less than three weeks after the channel was obtained it was closed up completely by the shifting sands of the bar; and this is what any marine engineer would have predicted.

The outer bar of Brunswick is a part of the great littoral cordon of the Atlantic Coast. The late General Q. A. Gillmore says of it:* "This bar is caused by waves and tides and not by river sediment." In 1876 there was 17 feet on it at low water, where now there is only about 13 feet. On a bar of this kind there are always several forces unceasingly and untiringly at work to perfect and build up the littoral cordon. These are the waves and currents. The littoral cordon exists in all seas, and is particularly developed on sandy and shingly coasts. The Italians call it "the zone of sounding up."

Thomas Stevenson† says, "Wherever the heaviest waves strike obliquely on the shore, the shingle, if there be any, travels across the beach and is very apt to fill up the entrances to harbors."

* See Report to the Chief of Engineers, United States Army (1876, I, 489).

* "Design and Construction of Harbor."

It is this same action that closed the harbor of Greytown, Nicaragua. The prevailing winds there are from the northeast, and the waves strike the beach obliquely. The beach is composed of coarse, dark colored volcanic sand, of low specific gravity, which does not compact or harden under the action of the waves, but remains soft. This sand is very easily moved, and it was carried along toward the west, in a zigzag manner, by the action of the waves, which strike the beach in the late summer and fall with great force, often sending up spray 20 feet high for days. The sand is carried alternately forward on the long hypothenuse of a right angled triangle, and then drawn down toward the sea by the reflux of the wave, to be again taken up and rolled forward on its diagonal course. In this way a tongue of sand was carried across the bay in a depth of over 40 feet of water, until it joined the western shore, and the littoral cordon was perfected, making a closed lagoon of the harbor.

Professor Henry Mitchell, Chief of Physical Hydrography on the United States Coast and Geodetic Survey, who examined this harbor in 1872 or 1873, estimated that the rate of deposit was 750 000 cubic yards per annum. He says :* "The littoral cordon is a developing physical phenomenon of the coast, distinguishing peculiarly, I think, the present geological period, and perhaps the most interesting and important among the changing features exhibited by our comparative surveys. * * * * * Upon our Southern seaboard it is the grand characteristic, and is almost continuous from Montauk to the coral reefs of Florida. and from St. Marks to Yucatan. * * * * * The tendency of the natural forces now operating upon the coast seems to be toward a complete smoothing away of all the indentures of the shore line."

As I said before, the agents that bring about this result are the waves and currents. Where there is a littoral current, this impinging against the shore tends to fill up all channels cut through the sandy cordon, and in this work the winds assist by producing or augmenting the current. On the east coast of Florida, during the winter, "northers" are of frequent occurrence. These winds are usually very strong for three days, and I have seen on the ocean beach, between Mosquito and Matanzas inlets, the well defined "rip" just outside the breakers, produced by the strong current flowing north, the wind at the same time blowing in the opposite direction, producing a banking up of the water against Cape Canaveral, and the current therefore ran against the wind to produce an equilibrium.

The waves act on the littoral cordon in two ways. Those waves that strike the bar diagonally travel along, and carry the sand with them, as I have described in the case of Greytown Harbor, filling up all indentures. Those great rollers that come in from the ocean, normal to the coast line, pile up the sand, gathered from the ocean floor, on the bar.

It is often the case that the flood tide follows a different channel from the ebb, especially on the first quarter. At Cumberland Sound, Ga.,

* Report United States Coast Survey, 1869, Reclamation of Tide Lands.

and St. John's Bar, Fla., and at numerous harbors and inlets on the New England coast, the tide first makes up inside channels along the shore, often at right angles to the ebb current. This action tends to fill up the ebb channels by deposits brought into them.

Lieutenant C. H. Davis,* U. S. Navy, shows how the flood tide takes up and ejects its burdens on the bars and beaches; but there is one phenomenon of the flood which he has not touched upon, and which I do not remember to have seen referred to by any one, and that is the difference in the character of the sand floor of the ocean or estuary during the ebb and during the flood. I first had my attention attracted to this when taking borings at Fort George Inlet, Fla., on surveys for the Florida Ship Canal. We found the sand of the beach, near low water, so hard and compact during the prevalence of the ebb tide, that with all our efforts we were unable to drive an iron rod down more than 2 feet; but after the tide turned and commenced to run up, the rod went down with little difficulty. During the ebb the sands of the beach, near low water, are also comparatively dry, and no water gathers in the foot tracks; but after the tide begins to rise, the whole bottom of the sea seems alive; the water is rising through the sand, and makes a little pool in every foot track, and the hard, compact sand floor changes to soft, yielding sand full of live water. I have noticed this fact also at the mouth of Chebacco River and Ipswich River, in Massachusetts. I took advantage of this fact in driving the iron pipes which I used for making the first ranges of the jetties which I laid out at St. John's Bar and also at Cumberland Sound. Essaying to start them before the tide turned, I found that a stalwart negro, with a heavy sledge, only succeeded in battering the top of the pipe, which could not be made to penetrate more than 2 or 3 feet; but when the tide began to rise and the sand became, as it were, alive, the pipes went down easily 6 and 8 feet. I judge from this, that the large storm rollers or ground swell, rolling in from the open sea at the first of the flood, will tear up and transport the sand from the ocean bed in shoal water, and throw it upon the littoral cordon, more readily than at any other time; and this, I think, accounts for the milky and soily character of the flood water, so often seen inside the bars.

The quotations in regard to the littoral cordon that I have given are not mere hypothesis. They are borne out amply by experience and experiment, and many more could be added. For years, however, in the face of nature and experience, efforts have been made to open and maintain cuts or channels through the littoral cordon by dredging and scraping. At St. John's Bar, Fla., $57 475.28 were expended in this desultory warfare before the construction of the jetties was commenced, and with absolutely no results, the channels formed filling up in a

* See his memoir on the "Law of Deposit of the Flood Tide," published in Smithsonian Contributions to Knowledge.

few weeks. At Nantucket Harbor, at New York Harbor, at the mouth of the Mississippi, and in fact at nearly every harbor on the Atlantic coast, this experiment has been repeatedly tried and with only one result—failure.

The Board of Trade of Brunswick might have saved their time and money; for how could they hope to succeed in the face of such a multitude of adverse facts? This method, however, might be made available and prove very advantageous for loosening and dispersing a hard sand shoal between jetties or training walls, where the current was strong enough to bear the displaced sand into deeper water; but in an unprotected cut it could be of no value, as these experiments and the experience of years have conclusively proved. It might also be used, instead of a dredge, for excavating a narrow channel or basin in the still water of an inner harbor, and much cheaper and more expeditiously; but this application would be limited to a certain depth of water, I should judge, as in comparatively shoal water the explosion could not fail to produce a crater of more or less width and depth.

Mr. Reed, M. Am. I. M. E.—Hydraulic dredges, in which the material is elevated by means of an ascending column of water, form a class distinct from that which includes bucket, dipper and clam shell dredges. The suction dredger, in which the centrifugal pump at the top of the column actuates the same, is, I believe, the only representative of the hydraulic class which has survived in practical use, though it is by no means the only style proposed or attempted. Inventors have also experimented with exhausted air chambers at the top of the tube, or with injector devices, either of water or of steam, at the foot of the tube. The sand pump used by Captain Eads in sinking the piers of the St. Louis Bridge and devised by him, employed the injector principle, an annular jet of water being used. General Roy Stone experimented in 1885, on dredging in Gedney's Channel, with an injection or induction tube having two round water jets entering the tube near its foot and directed upward. The results were unsatisfactory, though I believe some success had previously been attained with this apparatus in dredging other material than sand.

In connection with hydraulic dredges, the scraper at the base of the tube, by which the material is brought into the ascending column as the lower end of the tube is drawn along over the bottom, has been the subject of much invention and experiment. The well known boring property of a jet of water has been a favorite device for this purpose on hard bottoms. Captain Eads, in 1877, took out patents for a system of water jets attached to the scraper or plow of a suction dredger. A centrifugal pump actuated the dredging column, while force pumps actuated the stirring jets. I do not know whether this principle is utilized or not in actual practice at the present day. Suction dredgers are now used, however, extensively, notably in New York Harbor work, their great

advantage being, I believe, their capability of working at a considerable depth in a moderate seaway.

The paper just read illustrates a favorite fancy periodically revived, namely, to stir up the bottom during the ebb tide, with the expectation of the material being carried out in suspension. General Stone devised for that purpose another apparatus which he called a hydraulic plow. It consisted of a large pipe with the lower end bent upward, from which was discharged a powerful jet of water. This was dragged along the bar to be dredged, stirring up the sand and throwing it high up into the ebb current, which carried it a greater or less distance out. No practical results of value seem to have ensued, however, though the apparatus was tried quite extensively on the bar in New York Harbor in 1884.

I believe successful results were obtained some years ago on some harbor on the Pacific coast by stirring up the bottom by means of the propeller of a steamer anchored in the ebb tide, which, as it were, followed its work out as one would sweep out a room.

Having had occasion to investigate this subject some years ago, it occurred to me that the principles involved in ore dressing, with which I was familiar as a mining engineer, might be introduced to determine theoretically how far a horizontal current of water of a given velocity would transport material of a given kind in suspension before it would settle, starting from a given height. I found that the formulas worked out by Rittinger in his classic "Aufbereitungskunde," and by Von Sparre in his "Theorie der Separation," might be applied here with considerable success. The same formulas could also be used to determine the velocity of up-flow necessary in a suction dredger to carry up any given material at a given rate.

In ore dressing, the mineralized rock after being crushed consists of a great variety of particles—some of rock, some of this mineral, some of that, and ranging in size from the coarsest to fine dust or slime. The problem then is to separate out the barren rock, and to classify, if possible, the mineral particles into their various kinds; so that, for example, galena, pyrites and zinc blende will be obtained, each as far as possible free from the other. The apparatus used, such as jigs, "spitzkasten" and "spitzlutte," shaking and percussion tables, vanners, buddles, and so on, all depend upon, as fundamental principles, either the varying rate of subsidence of different kinds of particles in suspension in water, or the varying rate of rolling or sliding along a surface, of particles exposed to a horizontal current of water. It is the former of these principles which applies mainly to our case.

Rittinger's formula* for round particles settling from suspension in a horizontally moving stream of water is

$$x = \frac{y\ C}{\sqrt{\frac{d\ (\delta - \Delta)}{\Delta} \cdot \frac{2\ \gamma}{3\ \alpha_3\ \Delta}}}$$

* All distances are to be taken in meters.

in which x = the horizontal distance to which a given particle will move before settling.

y = the depth of water through which it must settle—starting from rest.

d = the diameter of the particle.

δ = its specific gravity.

C = the horizontal velocity of the stream.

Δ = the specific gravity of the liquid, in the case of sea water say 1 026.

γ = the weight of 1 cubic meter of water in kilogrammes = 1 000.

α_3 = the pressure in kilogrammes exercised upon a sphere of one square meter principal sectional area by a column of water at a velocity of 1 meter per second. This factor is computed at 25.5.

It will be seen that both the size and the specific gravity of the particles must be taken into consideration, and that a miscellaneous assemblage of particles discharged or thrown up into a horizontally moving stream at a given height above the bottom will be deposited down stream in classified bands at right angles to the current. Any special band may contain, mingled together, certain heavy, fine particles and certain light, coarse particles. If the current be sufficiently swift to keep some particles rolling or moving after they reach the bottom, the result will be more complicated. Rittinger's formula on that subject might perhaps be applied then, were it not for the fact that the bottom will not be a plane nor a hard surface.

For irregularly shaped particles a co-efficient is introduced in the formulas which modifies the results somewhat. With ordinary sea sand the particles will be, say, quartz, the coarsest; felspar, medium; hornblende, finest. In silt will be found particles of slaty or calcareous mineral largely. I add one of Rittinger's tables and a few references on the subject of ore dressing.

Upward Velocities in meters per second of stream of fresh water necessary to hold in suspension different particles.

Material.	Specific Gravity.	Greatest Diameter of Particles in Millimeters.							
		10	8	6	4	3	2	1	½
Galena	7.5	.62	.55	.48	.39	.34	.20	.20	.14
Pyrites	5.	.49	.43	.38	.30	.26	.22	.15	.10
Quartz	2.6	.30	.27	.24	.19	.17	.14	.10	.07
Coal	1.3	.13	.12	.10	.08	.07	.06	.04	.03

See Rittinger, "Aufbereitungskunde," and latest supplement (not translated).

Von Sparre, "Theorie der Separation" (not translated).
Prof. H. S. Munroe, of Columbia College, "On the Movement of Solid Bodies in Water," *School of Mines Quarterly*, January, 1888.
Also list of works on ore dressing, by Prof. Munroe, in the same periodical, January, 1889.
Also papers of the American Institute of Mining Engineers on the works at Clausthal and Pribram, by J. C. F. Randolph, Vol. VI; Ellis Clark, Vol. IX.
Also Kunhardt's "Ore Dressing in Europe," published by Wiley.

Captain O. M. Carter, M. Am. Soc. C. E.—The writer's original paper having been misapprehended by some of the gentlemen participating in the discussion, it may be well, for their benefit, to reiterate some of the facts stated therein and to give some of the reasons which led to the presentation of the paper.

No money has ever been appropriated by Congress for the improvement of the ocean bar at Brunswick, Ga., and no work other than the execution of surveys has ever been done there by the United States or under the direction or control of any officer or agent thereof. When the bar had shoaled to such an extent as to seriously interfere with the commerce of the port, the City of Brunswick appropriated $5 000 for the purpose of temporary improvement, and the writer was consulted as to the best method of expenditure. The sum appropriated was so small that any attempt at permanent improvement was out of the question. The distance across the bar between the inner and outer 15-foot mean low water contours was only from 650 to 750 feet, and it was possible that if a channel were cut through the narrow shoal it might remain open for a few months, perhaps longer, and so afford a slight relief to commerce—at least permit some heavy laden vessels, which had been waiting for weeks for water enough to cross the bar, to go to sea.

The methods recommended by the writer were, as previously stated, dredging or sluicing. The exposure of the work would not permit the use of any but pump dredges with flexible suction pipes, and none could be hired at a reasonable rate for so small an amount of work. Propeller sluicing was recommended as an alternative method, and as a last resort, sluicing by water jet. Only water-ballast steamers are suitable for propeller sluicing on a bar with as great a tidal range as at Brunswick, and as no such steamer available for work was then in port, the committee, rather than wait for the arrival of a steamer, decided to experiment with high explosives, although it was informed that failure would probably be the result.

Shortly after the work was completed, newspapers on the Atlantic and Gulf coasts began to give glowing accounts of the success of the "new methods of harbor improvement," which, it was claimed, were much more successful and much less expensive than those approved

by the Corps of Engineers, inasmuch as by the use of high explosives, to quote the language of one newspaper: "What would cost the Government millions may be effected at an expense of a few thousand dollars." As a proof of the success of the work it was claimed that a vessel drawing 20 feet and 9 inches had gone to sea through the new channel.* The writer then made a survey of the bar, which showed, as was expected, no appreciable gain in depth as a result of the work done. Letters from a number of civil engineers asking about the "new methods of harbor improvement," led the writer to present the facts in the case to the society. No difficulty would have been experienced in excavating a channel through the bar with pump dredges, such as are used in New York Harbor and referred to by Mr. Collingwood, and that method of temporary improvement was recommended by the writer; but it was found to be impossible, with the funds available, to hire a suitable dredge at that time.

Replying to the criticism of Mr. Lewis, the writer is informed by the diver who did the work that the charges were sunk to depths varying from 1 to 5 feet, and that the best results were obtained with depths of from only 1 to 2 feet.

Mr. Paret's statement that soundings on the ocean bar 4 or 5 miles from shore can, as a rule, hardly be relied upon to give results within half a foot of the proper depth, is of course correct; but it is customary to sound to feet and tenths, and the results were given as obtained, without intending to claim for them absolute correctness. The writer agrees with Mr. Paret as to the value of the directions and velocities of the currents at the time of the experiments, but unfortunately no such record was kept. Mr. Paret is in error in stating that a somewhat similar experiment to sluicing with a propeller or water jet was made at Brunswick two years ago. A harrow was dragged across the bar, and gave, as stated by the writer, no sensible increase in depth. Sluicing has never been tried on Brunswick Bar, nor has propeller sluicing ever been tried, as far as the writer is aware, on a bar with any considerable swell; but there is no reason why it should not be conducted wherever dredging is practicable. Descriptions of the successful removal of bars on the Pacific Coast by propeller sluicing, referred to by Mr. Reed, will be found in the annual reports of the Chief of Engineers for 1882–85.

Mr. Haupt inquires how it happens "that this report is published in advance of the annual official report of the Chief of Engineers." The writer's official conduct and his relation to the Chief of Engineers are matters which cannot properly concern Mr. Haupt in the least, but

*That the taking to sea through the so-called "new channel" a vessel drawing 20 feet and 9 inches on high water of spring tides, is no proof of an increase in depth, was shown by the writer in his original paper, and Mr. Haupt's reiteration of that statement is, of course, correct.

for his information it may be stated that the official report of the Chief of Engineers deals with work carried on under the direction of the Engineer Department, and as the work in question was something with which that department had nothing whatever to do, the writer's paper could, of course, have no proper place in that report.

It is difficult to understand how Mr. Haupt ascertains that "the function of the Government, as represented by its official, appears merely to have been to report the results unofficially through this society for the benefit of the profession," when the Government had nothing whatever to do with the matter, and when it is distinctly stated in the original paper that the funds were provided by the City of Brunswick and disbursed under the direction of a committee of the Board of Trade, in accordance with its own plans.

Mr. Haupt evidently confuses the work with dredging, in which material is entirely removed, when he states that "*the modus operandi* was defective in that the cut was begun on the inner slope of the bar instead of the outer." Where the material composing a bar is simply thrown into suspension and pushed along by the tidal currents, work carried on during the ebb tides should be begun on the inner slope of the bar. To use an illustration with which Mr. Haupt may be more familiar than with harbor work, it would be better in sweeping a room to begin at the point farthest from the door through which the sweepings are to be borne, than to begin at the door and proceed in the other direction. The amount of material to be removed not being increased by that order, a little reflection will doubtless show him that it will not increase the cost of removal.

The interesting difference in the character of the sand floor of the ocean on the ebb and on the flood, referred to by Mr. LeBaron, is discussed in an official report on the survey of the bar at Brunswick, recently made under my direction by Lieutenant Thomas H. Rees, Corps of Engineers, but which has not yet been published. Mr. LeBaron is in error in stating that the method of excavation by high explosives might "be used instead of a dredge for excavating a narrow channel or basin in the still water of an inner harbor, and much more cheaply and expeditiously," inasmuch as the only case in which it could be used successfully in material of the character described, instead of a dredge, would be in running water and in a current strong enough to carry away the material thrown into suspension. Should he have intended "still water" to mean smooth water, as contradistinguished from the rough water of an ocean bar, it may be stated that at points where the water is considerably agitated and where strong currents prevail there is more probability of the material's being carried away than there is in smooth water where no such agitation takes place.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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509.
(Vol. XXV.—November, 1891.)

STRESSES IN RAILWAY BRIDGES ON CURVES.

By WARD BALDWIN, M. Am. Soc. C. E.

WITH DISCUSSION.

The complete investigation of the effect produced upon the live load stresses in a bridge by the curvature of the track; by the accompanying super-elevation of the outer rail; and by the eccentricity of the center of the track with reference to the axis of the bridge, has not yet, so far as the author is aware, been published. While the solution of this problem involves only well known methods, it is thought that the general formulas, put in a convenient shape for reference, may be practically useful. That the effect referred to is of sufficient magnitude to demand consideration, even when the degree of curvature of the track is small, is made clear by a comparison of the live load stresses in the members of the inner and outer trusses of the bridge, of which Fig. 1 is the strain sheet for the live load. This bridge, which was recently erected under the supervision of the author, is 187½ feet long, with the track on a 3-degree curve, and was designed by the Keystone Bridge Company. The stresses marked above the chords and diagonals, and on the left of

Fig. 1.

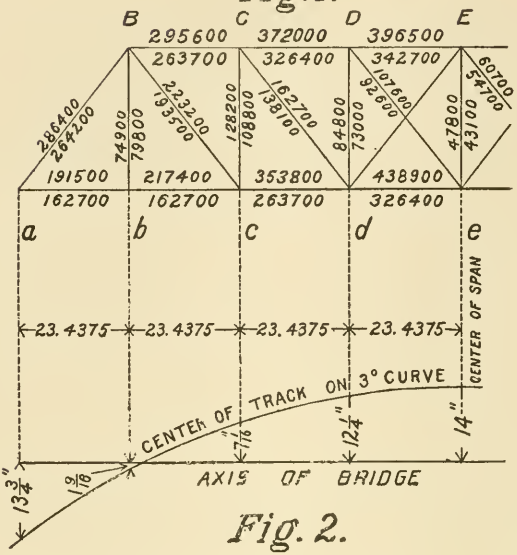
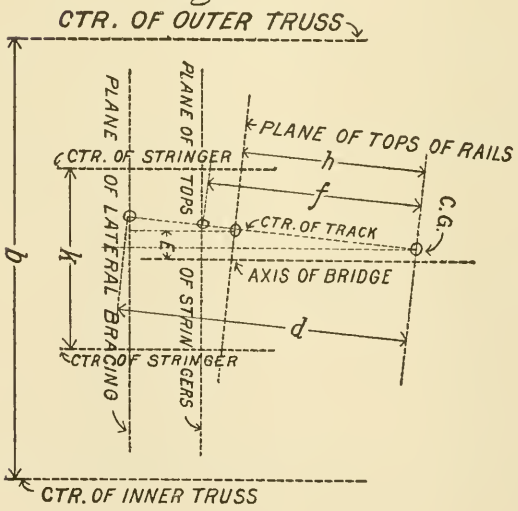


Fig. 2.

Fig. 3.



the verticals are the live load stresses on the outer truss; the other stresses are produced by the live load on the inner truss. The location of the center of the track with reference to the axis of the bridge is shown on an exaggerated scale in Fig. 2. There are two lines of stringers spaced 7½ feet apart on centers, and the axis of the stringers coincides with the axis of the bridge.

First, consider the general case shown in Fig. 3.

Suppose, that for a given position of the train we have at any given panel point, a panel live load "P," for both trusses, and suppose that the track is on a curve of radius "R." In Fig. 3 let the point CG be the center of gravity of the panel live load "P" (this point is taken to be on a normal to the plane of the track through the center of the track, and at a height above the track of the center of gravity of the train); let the super-elevation of the outer rail be s; let the angle of the plane of the track with the horizontal be A, so that sin. $A = \frac{s}{g}$, where g = gauge of track; let E_1 be the eccentricity, or horizontal distance from the axis of the bridge to the center of the track; the positive direction for E_1 being from the axis of the bridge toward the outside of the curve; let h, d and f (all measured in a direction perpendicular to the plane of the track) be the respective distances from CG to the plane of the track, to the plane of the lateral bracing, and to the plane of the tops of the upper flanges of the stringers. Let b be the distance center to center of trusses; k the distance center to center of stringers; and V the velocity of the train in feet per second. Then the centrifugal force of the panel load P is $\frac{PV^2}{32.2R} = PF$; where $F = \frac{V^2}{32.2R}$.

The forces at CG are the centrifugal force PF, considered as acting horizontally, and the panel load P, acting vertically. These two forces constitute the exterior forces acting on the bridge at the panel point considered. These forces must be balanced by the interior stresses induced by them in the trusses, and in the lateral bracing of the bridge.

The centrifugal force will form a couple with the induced shear in the lateral bracing; the moment of this couple being,

$$PFd \cos. A \quad \ldots\ldots (1)$$

This couple increases the load on the outer truss by

$$\frac{1}{b}(PFd \cos. A) \quad \ldots\ldots (2)$$

and decreases the load on the inner truss by the same amount.

The part of the panel load that is carried by the outer truss is

$$\left(\frac{b}{2} - h \text{ sin. } A + E_1\right)\frac{P}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and the part carried by the inner truss is

$$\left(\frac{b}{2} + h \text{ sin. } A - E_1\right)\frac{P}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The total outer panel load is found by combining equations (2) and (3) to be

$$\frac{P}{b}\left(\frac{b}{2} - h \text{ sin. } A + E_1 + Fd \text{ cos. } A\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The total inner panel load is found, by combining equations (4) and (2) to be

$$\frac{P}{b}\left(\frac{b}{2} + h \text{ sin. } A - E_1 - Fd \text{ cos. } A\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Now, the inner panel load has its greatest value when the second member of equation (6) is zero. This will be the case when the train stands still, for then F will be zero. The maximum inner panel load occurs, therefore, when the train stands still, if impact is neglected; and is found from equation (6), by putting $F = O$, to be

$$\frac{P}{b}\left(\frac{b}{2} + h \text{ sin. } A - E_1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The outer panel load, as appears from a consideration of equation (5), has its maximum value when F has its greatest value.

The general formulas are as follows, referring to equations (5) and (7):

Horizontal panel load on lateral bracing

$$P\,F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

Maximum inner panel load

$$\frac{P}{2}\left[1 + 2\left(\frac{h \text{ sin. } A - E_1}{b}\right)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

Maximum outer panel load

$$\frac{P}{2}\left[1 - 2\left(\frac{h \text{ sin. } A - E_1}{b}\right) + \frac{2\,Fd, \text{ cos. } A}{b}\right] \quad \ldots\ldots\ldots (D)$$

The application of these formulas to any special case may be simplified by the following considerations:

In equation (A) the factor F depends on the speed of the train, and the curvature of the track. This factor is a constant for all panel loads in any bridge, and needs to be determined but once. The factor $2\left(\frac{h \text{ sin. } A - E_1}{b}\right)$ in equations (B) and (D) depends on the elevation

of the outer rail, and on the eccentricity of the loading at each panel point. This factor must be determined, therefore, for each panel point. The factor $\left(\frac{2\,Fd\,\cos.\,A}{b}\right)$ in equation (D) depends on the elevation of the outer rail; on the speed of the train, and on the curvature of the track. This factor is, therefore, a constant for all panel loads, and needs to be determined but once.

Effect on Trusses.—In using these formulas to find the stresses in the truss members, the method of procedure is to find the position of the loading that will produce the maximum stress in the particular member considered, in the same way as if the bridge were on a straight line; then find all the panel loads for this position of the loading; then by means of equations (B) and (D) find the actual panel loads on the inner and outer trusses; then use these panel loads to find the stress in the given member. The value of E_1 in these equations may be taken to be the average value of E_1 in the two panels contiguous to the panel point considered.

As the maximum inner panel load is found for the train standing still, the usual allowance made for impact, if applied to the stresses on the inner truss, will give relatively greater strength for the inner truss than for the outer truss. It is, however, an easy matter to compare the increase of stress due to the specified impact with the decrease of stress due to centrifugal force, given by the last term, of equation (6), if this refinement is thought desirable. The error in adding the impact allowance to the stresses found by using equation (B) will be on the safe side, and it has been the practice of the author to ignore it.

Stringers.—The division of the loading between the stringers in any panel is governed by the same conditions as the division of a panel load between the outer and inner trusses, provided, first, that there are only two track stringers, and that the tops of the stringers are in one horizontal plane; and, second, that the eccentricity of the track with respect to the axis of the stringers is the same throughout the length of each stringer. The first condition is supposed to be complied with in this discussion, as this is usually the case, although some engineers secure the elevation of the outer rail by framing the stringers out of level. The second condition is never realized, but the eccentricity of the track may be taken at the average eccentricity for the stringer considered,

with a close approximation to the truth. Supposing, then, that the tops of the stringers are on the same level, and that E_2 is the average value of the eccentricity of the track with respect to the axis of the stringers for the given panel; the equations expressing the division of the loading between the inner and outer stringers may then be written directly from equations (B) and (D) by substituting k for b, and f for d in those equations. Let L be the total load in the panel, then the load on the inner stringer is,

$$\frac{L}{2}\left[1+\frac{2\,(h\sin.\,A-E_2)}{k}\right]\ldots\ldots\ldots\ldots\ldots\ldots\text{(E)}$$

and the load on the outer stringer is

$$\frac{L}{2}\left[1-2\left(\frac{h\sin.\,A-E_2}{k}\right)+\frac{2Ff\cos.\,A}{k}\right]\ldots\ldots\ldots\text{(F)}$$

These equations (E) and (F) also, evidently, give the relation between the maximum stresses in the stringers of any panel for an equal division of the loading between the two stringers, and for the actual division of the loading as determined by the centrifugal force, eccentricity, etc. The actual maximum bending moment and shear for each stringer can be conveniently found, by first finding the maximum bending moment and shear on the stringers when the track is on a straight line, and the stringers are symmetrically placed with reference to the axis of the track; then multiply these stresses by the values of the multiplier of $\frac{L}{2}$ in equations (E) and (F) for each panel, to get the actual stresses.

It is seen from equation (A) that there is also a horizontal shear and bending in the plane of the tops of the stringers, which at any point is equal to the vertical shear or bending at this point, due to the total live load in the panel, multiplied by F. It thus appears that the vertical and horizontal stresses in the stringers can all be found for the case when the track is on a curve by multiplying the corresponding stress in the stringer of a bridge on a straight line by a certain factor.

As the panel length is usually uniform for a given bridge, it is in most cases only necessary to find the maximum bending and shear for one panel, supposing the bridge to be on a straight line, and then find the values of the actual stresses by multiplying by the values of the multiplier of $\frac{L}{2}$ in equations (E) and (F) for each panel. The horizontal stresses in the plane of the tops of the stringers may be resisted by the

stringers acting as independent beams; or by the bracing of the top flanges of the stringers; or by the main laterals, when they are in the plane of the tops of the stringers and are attached to the stringer flanges.

Floor-beams.—When, as in the case of a bridge on a straight line, the center of gravity of the loading, the axis of the stringers, and the center of the floor-beam are all in the same vertical plane, the loading is divided equally between the inner and outer stringers ; the greatest live load shear on the floor-beam is equal to the panel load on either the inner or outer truss; and the greatest live load bending moment on the floor-beam is equal to the product of this panel load multiplied by the distance from the center of either truss to the nearer stringer; and if, as is usual, the panel lengths are uniform, these stresses will be the same for all floor-beams.

Now in the case of the track being on a curve, the loading in a panel is not in general equally divided between the stringers, but the effect of eccentricity, centrifugal force, etc., is to change the proportionate division of the loading between the stringers, one stringer having its load increased, and the other stringer having its load decreased by the same amount. The total load on both stringers being, however, the same as when the track is on a straight line, the effect on the floor-beam of this unequal loading of the stringers in a panel is equivalent to that of a couple with vertical forces equal to the increase or decrease of the stringer panel load, and with an arm equal to the distance from center to center of stringers. This couple is balanced by the stresses induced by it in the trusses, the arm of this resisting couple being the distance center to center of trusses.

Two cases may occur, viz.:

First.—Each line of stringers may be in one continuous straight line, in which case the axes of the stringers and bridge are coincident.

Second.—Each line of stringers may be in a broken line, conforming more or less closely with the curve of the track, in which case the axis of the stringers in any given panel does not usually coincide with the axis of the bridge. Each stringer is, of course, parallel to the axis of the bridge.

In either one of these cases, the reaction of each stringer may be found by applying equations (E) and (F); and the stresses in the floor-

beams are then readily determined. The calculations, may, however, be more conveniently made as follows:

First.—Suppose the stringers to be in a straight line, and that the axis of the stringers coincides with the axis of the bridge. If the eccentricity of the track with reference to the axis of the stringers, in the two panels contiguous to the floor-beam considered, be taken at a uniform average value of E_1, then the maximum stringer panel-load at the given floor-beam is found by analogy from equations (B) and (D) to be

$$\text{For the inner stringer, } \frac{P}{2}\left[1+\frac{2\,(h\sin.\,A-E_1)}{b}\,\frac{b}{k}\right] \quad \text{(H)}$$

$$\text{For the outer stringer, } \frac{P}{2}\left[1-\left[\frac{2\,(h\sin.\,A-E_1)}{b}-\frac{2\,Ff\cos.\,A}{b}\right]\frac{b}{k}\right] \quad \text{(I)}$$

That is, if the train stands still, the inner stringer panel-load is greater than for a bridge on a straight line by

$$\frac{P}{2}\,(h\sin.\,A-E_1)\,\frac{2}{k} \quad \text{(J)}$$

and the outer stringer panel-load is less than for a bridge on a straight line by the same amount. Also, if the train moves at the maximum velocity, the outer stringer panel-load is greater than for a bridge on a straight line by

$$\frac{P}{2}\left[\frac{2.Ff.\,\cos.A}{k}-\frac{2\,(h\sin.\,A-E_1)}{k}\right] \quad \text{(K)}$$

and the inner stringer panel-load is less than for a bridge on a straight line by the same amount.

Since the greatest bending in the floor-beam is near the stringer which has the greater panel-load, the values of (J) and (K) show at once near which stringer the greatest bending in the floor-beam occurs. If (J) is greater than (K), the greatest bending is near the inner stringer; if (J) is less than (K), the greatest bending is near the outer stringer. The stringer panel-loads may be found by using (H) and (I); and are needed in finding the bearing value in the floor-beam web of the rivets attaching the stringers. The values of (H) and (I) may evidently be quickly found from (B) and (D).

The other stresses in the floor-beams are conveniently found by first calculating what the stresses would be if the track were straight, and the loading were equally divided between the stringers; and then applying as corrections to these stresses the changes therefrom caused by curvature, etc., as determined from equations (B) and (D). This is readily

done, since the stresses in the floor-beam at any point between a stringer and the nearer truss are proportional to the truss panel-load of that truss.

Second.—Suppose the stringers to be parallel to the axis of the bridge, but not to be in a straight line. In this case, the stresses in the floor-beams can be conveniently found by finding the truss panel-loads by means of equations (B) and (D); and then using these truss panel-loads to determine the stresses in each beam. The reactions at the ends of the stringers may be found by applying equations (E) and (F) to each stringer. In this case the greatest stresses in the floor-beam are not necessarily at the stringer having the greater panel-load.

The average values of E are found in all cases by taking the curve of the track as a polygon whose sides are the chord of the curve in each panel.

Numerical Example.—The application of these formulas to the special case of the bridge shown in Figs. 1, 2 and 3 may, perhaps, make their use better understood.

The data for the bridge are as follows:

$S = 3.1$ inches. $\quad F = \left(\frac{(58\frac{2}{3})^2}{32.2 \times 1910.1}\right) = 0.056$

$g = 56.5$ inches.

$h = 60$ inches. $\quad$ Sin. A $= \left(\frac{3.1}{56.5}\right) = 0.056$

$d = 78$ inches. $\quad$ Cos. A $= 0.998$.

$f = 78$ inches. $\quad R = 1910.1$ feet.

$b = 18.5$ feet $= 222$ inches. $\quad k = 7.5$ feet $= 90$ inches.

$V = 58\frac{2}{3}$ feet per second $= 40$ miles per hour.

$$E_1 \text{ for } b = \frac{-13\frac{3}{4} - 1\frac{9}{16} \times 2 + 7\frac{1}{16}}{4} = -2.45 \text{ inches.}$$

$$E_1 \text{ for } c = \frac{-1\frac{9}{16} + 7\frac{1}{16} \times 2 + 12\frac{1}{4}}{4} = +6.2 \text{ inches.}$$

$$E_1 \text{ for } d = \frac{7\frac{1}{16} + 12\frac{1}{4} \times 2 + 14}{4} = +11.4 \text{ inches.}$$

$$E_1 \text{ for } e = \frac{12\frac{1}{4} + 14 \times 2 + 12\frac{1}{4}}{4} = +13\frac{1}{8} \text{ inches.}$$

$$E_2 \text{ for } ab = \frac{-13\frac{3}{4} - 1\frac{9}{16}}{2} = -7.67 \text{ inches.}$$

$$E_2 \text{ for } bc = \frac{-1\frac{9}{16} + 7\frac{1}{16}}{2} = +2.75 \text{ inches.}$$

$$E_2 \text{ for } cd = \frac{7\frac{1}{16} + 12\frac{1}{4}}{2} = +9.66 \text{ inches.}$$

$$E_2 \text{ for } de = \frac{12\frac{1}{4} + 14}{2} = +13\frac{1}{8} \text{ inches.}$$

Equation (A) has the value $P \times 0.056$.

Equation (B) has values as follows :

$$\text{At } b.... \frac{P}{2}\left[1 + 2\left(\frac{3.36 + 2.45}{222}\right)\right] = \frac{P}{2}\left[1 + 0.052\right].$$

$$\text{At } c.... \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 6.2}{222}\right)\right] = \frac{P}{2}\left[1 - 0.026\right].$$

$$\text{At } d.... \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 11.4}{222}\right)\right] = \frac{P}{2}\left[1 - 0.072\right].$$

$$\text{At } e.... \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 13.12}{222}\right)\right] = \frac{P}{2}\left[1 - 0.088\right].$$

Equation (D) has values as follows:

$$b.... \frac{P}{2}\left[1 - 0.052 + \frac{2 \times 0.056 \times 78 \times 0.998}{222}\right] = \frac{P}{2}\left[1 - 0.013\right].$$

$$\text{At } c.... \frac{P}{2}\left[1 + 0.026 + 0.039\right] = \frac{P}{2}\left[1 + 0.065\right].$$

$$\text{At } d.... \frac{P}{2}\left[1 + 0.072 + 0.039\right] = \frac{P}{2}\left[1 + 0.111\right].$$

$$\text{At } e.... \frac{P}{2}\left[1 + 0.088 + 0.039\right] = \frac{P}{2}\left[1 + 0.127\right].$$

Equation (E) has values as follows:

$$\text{At } b.... 1 + \frac{2\,(3.36 + 7.67)}{90} = 1 + 0.245.$$

$$\text{At } c.... 1 + \frac{2\,(3.36 - 2.75)}{90} = 1 + 0.014.$$

$$\text{At } d....1 + \frac{2\,(3.36 - 9.66)}{90} = 1 - 0.140.$$

$$\text{At } e....1 + \frac{2\,(3.36 - 13.12)}{90} = 1 - 0.217.$$

Equation (F) for b becomes $1 - 0.245 + \frac{2 \times 0.056 \times 78 \times 0.998}{90} = 1 - 0.148.$

" " " c " $1 - 0.014 + 0.097 = 1 + 0.083.$

" " " d " $1 + 0.140 + 0.097 = 1 + 0.237.$

" " " e " $1 + 0.217 + 0.097 = 1 + 0.314.$

" (J) " b " $\frac{P}{2}\,(3.36 + 2.45)\,\frac{2}{90} = + \frac{P}{2} \times 0.130.$

" " " c " $\frac{P}{2}\,(3.36 - 6.2)\,\frac{2}{90} = - \frac{P}{2} \times 0.063.$

" " " d " $\frac{P}{2}\,(3.36 - 11.4)\,\frac{2}{90} = - \frac{P}{2} \times 0.179.$

" " " e " $\frac{P}{2}\,(3.36 - 13.12)\,\frac{2}{90} = - \frac{P}{2} \times 0.217.$

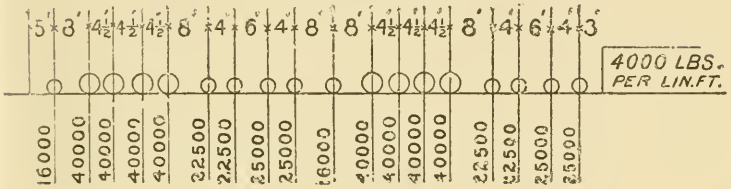
Equation (K) for b becomes $\frac{P}{2}(0.097 - 0.130) = -\frac{P}{2} \times 0.033.$

" " " c " $\frac{P}{2}(0.097 + 0.063) = +\frac{P}{2} \times 0.160.$

" " " d " $\frac{P}{2}(0.097 + 0.179) = +\frac{P}{2} \times 0.276.$

" " " e " $\frac{P}{2}(0.097 + 0.217) = +\frac{P}{2} \times 0.314.$

DIAGRAM OF LIVE LOAD.



STRINGERS.

The maximum live load moment on each stringer (supposing the loading to be equally divided between the two stringers), occurs under wheel 4 at $1\frac{1}{8}$ feet from the middle of the stringer; and is 293 000 foot-pounds. Now using the values of equations (E) as factors for the inner stringers, and of equations (F) as factors for the outer stringers, we find the maximum bending moments to be as follows:

Fig. 4.

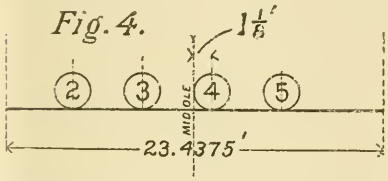


TABLE No. 1.

MAXIMUM MOMENTS FOR STRINGERS.

	M MAXIMUM FOR INNER STRINGER.	*M* MAXIMUM FOR OUTER STRINGER.
Stringer *ab*.....	293 000 x 1.245 = 364 800 foot-pounds.	293 000 x 0.852 = 249 600 foot-pounds.
" *bc*.....	" x 1.014 = 297 100 "	" x 1.083 = 317 300 "
" *cd*.....	" x 0.860 = 252 000 "	" x 1.237 = 362 400 "
" *de*.....	" x 0.783 = 229 400 "	" x 1.314 = 385 000 "

The maximum horizontal bending moment in the plane of the upper flanges is by equation (A) 293 000 x 2 x 0.056 = 32 800 foot-pounds. The main laterals are, in this bridge, attached to the upper flanges, and thus relieve the stringers from the greater part of this bending. The maximum live load shear at the end of the stringers is (supposing the

loading to be equally divided between the two stringers, and taking wheel (2) at the end of the stringer), equal to 57 900 pounds. Now, using equation (E) as a factor for the inner stringers, and equation (F) for the outer stringers, we find the maximum live load end shears for the stringers to be as follows:

Fig. 5.

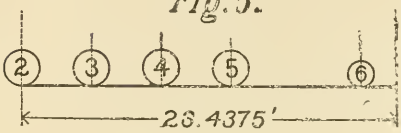


TABLE No. 2.

MAXIMUM END SHEARS FOR STRINGERS.

	S MAXIMUM FOR INNER STRINGER.	*S* MAXIMUM FOR OUTER STRINGER.
Stringer *ab*	57 900 x 1.245 = 72 100	57 900 x 0.852 = 49 300
" *bc*	" x 1.014 = 58 700	" x 1.083 = 62 700
" *cd*	" x 0.860 = 49 800	" x 1.237 = 71 600
" *de*	" x 0.783 = 45 350	" x 1.314 = 76 100

In this bridge the stringers are riveted to the floor-beam webs, and these end shears in Table No. 2 are used to determine the number of rivets required in the end of the stringer web, and the number of rivets required for single shear in the floor-beam web. The maximum panel load at each stringer is used to determine the number of rivets required for bearing in the connection to the floor-beam web.

The maximum panel live load for one stringer (supposing the loading to be equally divided between the two stringers) occurs with wheel 4 at a panel point, and is 75 900 pounds. Using equation (H) for the inner stringers and equation (I) for the outer stringers, we find the maximum panel live load to be as follows:

Fig. 6.

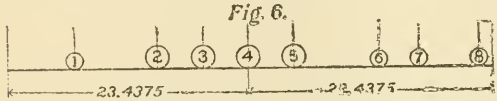


TABLE No. 3.

MAXIMUM PANEL LOADS FOR STRINGERS.

PANEL POINT.	PANEL LOAD.	
	Inner Stringer.	Outer Stringer.
b	75 900 x 1.130 = 85 800	75 900 x 0.967 = 73 400
c	" x 0.937 = 71 100	" x 1.160 = 88 050
d	" x 0.821 = 62 250	" x 1.276 = 96 800
e	" x 0.783 = 59 400	" x 1.314 = 99 700

Floor-beams.

The span of the floor-beams is taken the same as the distance center to center of trusses, namely 18.5 feet. The stringers are 7.5 feet center to center. If the bridge were on a straight line and the loading were equally divided between the stringers, the maximum live load bending moment on the floor-beam would be $75\,900\left(\frac{18.5-7.5}{2}\right) = 417\,450$ foot-pounds. In the present case the maximum live load moment occurs at the inner or outer stringer, according to which has the greater panel load. It appears, therefore, from the inspection of Table 3, that the maximum moment occurs at the inner stringer for floor-beam *b;* and at the outer stringer for the other floor-beams.

The correction for the floor-beam moment and shear is given by (B) for floor-beam *b;* and by (D) for the other floor-beams.

TABLE No. 4.

Maximum Bending Moments for Floor-beams.

Floor-beam.	*M* Maximum, Ft. lbs.
b	417 450 x 1.052 = 439 600
c	" x 1.065 = 444 600
d	" x 1.111 = 463 800
e	" x 1.127 = 470 500

TABLE No. 5.

Maximum Shear for Floor-beams.

Floor-beam.	*S* Maximum.
b	75 900 x 1.052 = 80 000
c	" x 1.065 = 80 800
d	" x 1.111 = 84 300
e	" x 1.127 = 85 500

The longitudinal compression of the upper flange of the floor-beams, and the tensile stresses in the lower chord of the outer truss that are produced by the horizontal component of the centrifugal force, may be computed with sufficient accuracy by using an equivalent uniform panel load. This uniform load is found for the position of the actual loading, giving the maximum stress in the end panel, and gives a uniform panel load of 121 600 pounds for the bridge, or of 60 800 pounds per truss. The

panel centrifugal force due to this panel load is, by equation (A), 121 600 $\times$ 0.056 = 6 810 pounds. The compression on the upper flanges of the floor-beams and the stresses in the laterals due to this horizontal panel load are easily found in the ordinary way, and so are not given here. The tensile stresses in the lower chord of the outer truss due to this horizontal panel load are as follows: The lateral diagonals of this bridge are stiff and form a double lattice system, so that the horizontal shear in each panel may be assumed to divide equally between the two diagonals, giving tension in one and compression in the other. This division of the laterals into two systems of course affects the chord stresses.

TABLE No. 6.

Tension in Lower Chord for Horizontal Force.

Chords.	Stress.
ab	6 810 x 1.75 x 1.267 = 15 100
bc	6 810 (1.75 x 3 — 0.5) 1.267 = 41 000
cd	6 810 (1.75 x 5 — 2) 1.267 = 58 200
de	6 810 (1.75 x 7 — 4.5) 1.267 = 66 900

These stresses are to be added to the tensile stresses in the lower chord of the outer truss that are produced by the vertical loading.

If the equivalent uniform loading of 121 600 pounds per bridge panel be used to find the stresses in the truss members, the work will be very much less than will be required to find the stresses for the actual loading. Both loadings will be used here so that a comparison of their relative accuracy may be made. For a uniform loading the method of procedure is as follows:

Equations (B) and (D) give the values of the panel loads on the two trusses as follows.

TABLE No. 7.

Panel Loads for Uniform Panel Load of 121 600 Pounds.

Truss.	Panel Point.						
	b	*c*	*d*	*e*	*f*	*g*	*h*
Inner	63 960	59 220	56 420	55 440	56 420	59 220	63 960
Outer	60 000	64 750	67 550	68 520	67 550	64 750	60 000

TABLE No. 8.

Reactions at Left Pier for Single Panel Loads.

Truss.	*b*	*c*	*d*	*e*	*f*	*g*	*h*	Total.
Inner.....	55 960	44 410	35 260	27 720	21 160	14 800	8 000	207 310
Outer.....	52 500	48 560	42 220	34 260	25 330	16 190	7 500	226 560

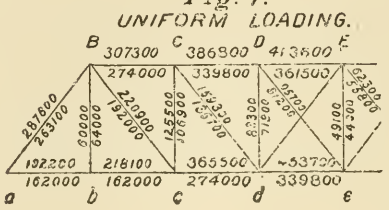
TABLE No. 9.

Stresses in Trusses for Uniform Panel Load of 121 600 Pounds.

Member.	Truss	Stress.	From Table No. 6, for Outer Truss.
ab.....	Inner.....	207 300 x 0.78 125 = 162 000	
bc.....	Outer.....	226 600 x " = 177 100	*ab* = 192 200
		350 600	*bc* = 218 100
cd.....	Inner	(207 300 x 2 — 64 000) 0.78 125 = 274 000	
		393 200	
BC.....	Outer	(226 600 x 2 — 60 000) 0.78 125 = 307 300	*cd* = 365 500
		434 800	
de.....	Inner.....	(207 300 x 3 — 187 100) 0.78 125 = 339 800	
		495 000	
CD.....	Outer	(226 600 x 3 — 184 800) 0.78 125 = 386 800	*de* = 453 700
		462 500	
DE	Inner	(207 300 x 4 — 366 700) 0.78 125 = 361 500	
		529 300	
	Outer	(226 600 x 4 — 377 100) 0.78 125 = 413 600	
aB.....	Inner	207 300 x 1.269 = 263 100	
	Outer	226 600 x 1.269 = 287 600	
		151 300	
cB.....	Inner	(207 300 — 56 000) 1.269 = 192 000	
		174 100	
	Outer	(226 600 — 52 500) 1.269 = 220 900	
		106 900	
dC.....	Inner	(207 300 — 100 400) 1.269 = 135 700	
		125 500	
	Outer	(226 600 — 101 100) 1.269 = 159 300	
		71 700	
eD.....	Inner	(207 300 — 135 600) 1.269 = 91 000	
		83 300	
	Outer	(226 600 — 143 300) 1.269 = 105 700	
		44 000	
fE.....	Inner	(207 300 — 163 300) 1.269 = 55 800	
		49 100	
	Outer	(226 600 — 177 500) 1.269 = 62 300	

These stresses are shown on the strain sheet in Fig. 7.

Fig. 7.



If the actual loading is used the method of procedure is as follows:

First the panel loads are found for each position of the loading that gives a maximum stress in a member. These positions of the loading can be determined graphically from a shear or load diagram with very little labor. The panel loads are then found by the formulas

$$P_n = \frac{M_{n-1} - 2\,M_n + M_{n+1}}{p},$$

where p is the panel length; P_n is the panel load at the panel point; and M_{n-1}, M_n, M_{n+1} are the sums of the moments of all the preceding loads about the $(n\text{-}1)^{th}$, n^{th}, $(n+1)^{th}$ panel points, respectively. This formula was first proposed by Mr. E. Thacher, M. Am. Soc. C. E.

TABLE No. 10.

PANEL LOADS PER TRUSS FOR BRIDGE ON STRAIGHT LINE.

MEMBER.	POSITION OF LOAD FOR MAXIMUM STRESS.	PANEL LOAD.						
		b	*c*	*d*	*e*	*f*	*g*	*h*
ab and *bc*	Wheel 4 at *b*	75 900	48 600	73 950	56 100	48 100	46 900	46 900
cd and *BC*	" 7 " *c*	67 000	56 650	62 900	64 750	49 600	49 900	46 900
de and *CD*	" 11 " *d*	72 700	51 400	68 950	60 900	48 900	46 900	46 900
DE	" 13 " *e*	30 850	74 000	48 000	75 900	53 500	46 950	46 900
aB	" 4 " *b*	75 900	48 600	73 950	56 100	48 100	46 900	46 960
cB	" 4 " *c*	17 300	75 900	48 600	73 950	56 100	48 100	46 900
dC	" 3 " *d*	0	8 100	72 600	51 500	68 600	61 100	48 950
eD	" 3 " *e*	0	0	8 100	72 600	51 500	68 600	61 100
fE	" 3 " *f*	0	0	0	8 100	72 600	51 500	68 600
Equation (B) for inner truss		1.052	0.974	0.928	0.912	0.928	0.974	1.052
Equation (D) for outer truss		0.987	1.065	1.111	1.127	1.111	1.065	0.987

The following table gives the values of the inner and outer panel loads to be used in finding the stresses in the corresponding members.

These values are derived from the above table by multiplying the respective panel loads by the multiplier of $\frac{P}{2}$ in equations (B) and (D).

TABLE No. 11.

Panel Loads on Inner and Outer Trusses for Actual Loading.

Member.	Truss.	Panel Loads.						
		b	*c*	*d*	*e*	*f*	*g*	*h*
ab	Inner	79 800	47 350	68 650	51 150	44 650	45 700	49 350
bc	Outer	74 900	51 750	82 150	63 200	53 450	49 950	46 300
cd	Inner	70 500	55 200	58 350	59 050	46 050	45 700	49 350
BC	Outer	66 100	60 350	69 900	72 950	55 100	49 950	46 300
de	Inner	76 500	50 050	64 000	55 550	45 400	45 700	49 350
CD	Outer	71 750	54 750	76 600	68 650	54 300	49 950	46 300
DE	Inner	32 450	72 100	44 550	69 200	49 650	45 750	49 350
	Outer	30 450	78 800	53 300	85 500	59 450	50 000	46 300
aB	Inner	79 800	47 350	68 650	51 150	44 650	45 700	49 350
	Outer	74 900	51 750	82 150	63 200	53 450	49 950	46 300
cB	Inner	18 200	73 900	45 100	67 450	52 050	46 850	49 350
	Outer	17 100	80 850	54 000	83 350	62 300	51 200	46 300
dC	Inner	0	7 900	67 400	46 950	63 650	59 500	51 500
	Outer	0	8 650	80 650	58 050	76 200	65 100	48 300
eD	Inner	0	0	7 500	66 200	47 800	66 800	64 300
	Outer	0	0	9 000	81 800	57 200	73 050	60 300
fE	Inner	0	0	0	7 400	67 400	50 150	72 150
	Outer	0	0	0	9 150	80 650	54 850	67 700

These panel loads are used to find the stresses in the truss members as given in Tables Nos. 12 and 13.

TABLE No. 12.

Fraction of Panel Load at Left Reaction.

Stress in Member.	Truss.	$\frac{7}{8}b$	$\frac{3}{4}c$	$\frac{5}{8}d$	$\frac{1}{2}e$	$\frac{3}{8}f$	$\frac{1}{4}g$	$\frac{1}{8}h$
ab	Inner	69 820	35 510	42 910	25 580	16 750	11 430	6 170
bc	Outer	65 540	38 810	51 350	31 600	20 050	12 490	5 800
cd	Inner	61 680	41 400	36 470	29 530	17 270	11 430	6 170
Bc	Outer	57 840	45 260	43 690	36 480	20 660	12 490	5 800
de	Inner	66 940	37 540	40 000	27 780	17 020	11 430	6 170
CD	Outer	62 780	41 060	47 880	34 330	20 360	12 490	5 800
DE	Inner	28 400	54 080	27 840	34 600	18 620	11 440	6 170
	Outer	26 650	59 100	33 310	42 750	22 290	12 500	5 800
aB	Inner	69 820	35 510	42 910	25 580	16 750	11 430	6 170
	Outer	65 540	38 810	51 350	31 600	20 050	12 490	5 800
cB	Inner	15 920	55 420	28 190	33 730	19 520	11 710	6 170
	Outer	14 960	60 640	33 750	41 680	23 360	12 800	5 800
dC	Inner	.....	5 930	42 120	23 480	23 870	14 900	6 440
	Outer	.....	6 490	50 410	29 030	28 570	16 280	6 040
eD	Inner	.....	.....	4 690	33 100	17 920	16 700	8 040
	Outer	.....	.....	5 630	40 900	21 450	18 260	7 540
fE	Inner	.....	.....	.....	3 700	25 270	12 540	9 020
	Outer	.....	.....	.....	4 580	30 240	13 710	8 460

TABLE No. 13.

Stresses in Truss Members due to Actual Loading.

Stress in Member.	Truss.	Left Reaction.	Stresses.		From Table No. 6 for Outer Truss.	
ab	Inner....	208 170 ×	0.78125 =	162 700	*ab*	191 500
bc	Outer....	225 640 ×	0.78125 =	176 350	*bc*	217 400
cd	Inner....	(203 950 ×	2 — 70 500) 0.78125 =	263 700	*cd*	354 000
Bc	Outer....	(222 200 ×	2 — 66 100) 0.78125 =	295 500		
de	Inner....	(206 880 ×	3 — {76 500 × 2 / + 50 050}) 0.78125 =	326 400	*de*	438 900
CD	Outer....	(224 700 ×	3 — {71 750 × 2 / + 54 750}) 0.78125 =	371 950		
DE	Inner....	(181 150 ×	4 — {32 450 × 3 / 72 100 × 2 / 44 550}) 0.78125 =	342 700		
	Outer....	(202 400 ×	4 — {30 450 × 3 / 78 800 × 2 / 53 300 × 1}) 0.78125 =	396 500		
aB	Inner....	208 170 ×	1.269 =	264 200		
	Outer....	225 640 ×	1.269 =	286 350		
cB	Inner....	(170 660 — 152 460	18 200) × 1.269 =	193 500		
	Outer....	(192 990 — 175 890	17 100) × 1.269 =	223 200		
dC	Inner....	(116 740 — 108 840	7 900) × 1.269 =	138 100		
	Outer....	(136 800 — 128 170	8 650) × 1.269 =	162 650		
eD	Inner....	(80 450 — 72 950	7 500) × 1.269 =	92 600		
	Outer....	(93 780 — 84 780	9 000) × 1.269 =	107 600		
fE	Inner....	(50 530 — 43 130	7 400) × 1.269 =	54 700		
	Outer....	(56 990 — 47 840	9 150) × 1.269 =	60 700		

These stresses are the ones given in the strain sheet in Fig. 1. A comparison of Figs. 1 and 7 will show the difference in the stresses found by these two methods to lie principally in the chords. Of course the floor system should be calculated for the actual loading in every instance.

In the bridge here discussed it will be noticed that the elevation of the outer rail is such that the centrifugal force of the train, *i. e.* (0.056 W*) is exactly balanced by the component of the weight of the train parallel to the track, *i. e.* (W. sin. $A = 0.056\ W$). This elevation of the outer rail is the one that should theoretically exist; and it is sometimes assumed that when this equality of centrifugal force and component of weight parallel to the track exists, the only effect of the centrifugal force will be to produce horizontal stresses in the laterals and chords of the

* This is really (0.056 W cos. A) = (0.056 $W \times 0.998$) = (0.056 W), appreciably.

loaded chord. The calculations here given, however, show that even when the rail has this elevation, nevertheless the stresses in the web members and unloaded chord are changed quite largely from what they would be for a bridge on a straight line. In many cases the outer rail is not elevated to correspond with the specified velocity; and then, of course, the effect of the curvature on the stresses will be somewhat changed from the case discussed. In fact in this case the elevation of the outer rail was made 2.25 inches, instead of 3.1 inches, so as to have the elevation of the track correspond with that on the two old spans already in place at the ends of this bridge.

Equalized Stresses.

The location of the track with reference to the axis of the bridge is of importance. In the case of a deck bridge this arrangement can usually be made so as to reduce the difference between the stresses on the two trusses to a minimum, which is a desirable thing to accomplish, as the variety of sections is thus reduced. In the case of a through bridge the necessary clearance, as well as the equalization of stresses, is an important factor of the problem. In through spans of moderate length, where the track is straight, it is customary to space the trusses just far enough apart to provide the specified minimum train clearance, as any greater width increases the lengths of the floor-beams, and of all the members of both lateral systems, and also of the transverse dimension of the piers. When the track is on a curve it may either be so located as to secure the minimum specified train clearance at the middle and ends of a through span, thus making the width between the trusses the least possible; or, it may be so located as to equalize as nearly as may be the stresses on both trusses. This latter arrangement will be secured at the expense of an increased amount of material in the floor-beams, laterals and piers, if a given clearance is to be maintained.

As the shifting of the track laterally on the bridge can only effect a transfer of part of the loading from one truss to the other, evidently the total metal in both trusses will not be changed, and there will be no economy secured by so locating the track as to equalize the stresses in the trusses, unless the resulting uniformity of dimensions will enable the contractor to reduce the price per pound.

In the case of the bridge discussed above, the position of the track for an equal clearance between the inner truss and the train at the ends of the span, and between the outer truss and the train at the middle of

the span, is as follows: Let E_a be the negative eccentricity of the center of the track with reference to the axis of the bridge at the ends, and E_b be the positive eccentricity of the center of the track with reference to the axis of the bridge at the middle of the span. The height from the tops of the rails to the bottom of the car is assumed to be 4 feet, and to the top of the car 14 feet. The length of a 34-foot car between truck centers is taken to be 28 feet. The super-elevation of the outer rail is $2\frac{1}{4}$ inches. The horizontal distance from the axis of the bridge to the axis of the floor of the car at the middle of the span is, then,

$$*E_b - 4\frac{2.25}{4.7} = (E_b - 2) \text{ inches} \ldots\ldots\ldots\ldots\ldots\ldots (M)$$

At the ends of the span the plane of the upper faces of the end posts cuts the plane of the top of the car at about the middle of the end panel, and the versine of a 28-foot chord of a 3-degree curve is about $\frac{5}{8}$ of an inch. Therefore the horizontal distance from the axis of the bridge to the axis of the top of the car at the middle of the end panel is

$$E_a + 14\frac{2.25}{4.7} + \frac{5}{8} - \frac{13\frac{3}{4} + 1\frac{9}{16}}{2} = \left(E_a - \frac{1}{4}\right) \text{ inches} \ldots\ldots (N)$$

Therefore, for an equal clearance at the ends and middle of the span we have $E_b - 2 = E_a - \frac{1}{4}$; therefore, $E_b - E_a = 1\frac{3}{4}$ inches. Actually $E_b - E_a = \frac{1}{4}$ of an inch, which is nearly correct.

If the super-elevation of the outer rail be taken at 3.1 inches, as used in the calculations, the value of equation (M) becomes

$$E_b - 4\frac{3.1}{4.7} = (E_b - 2\tfrac{1}{2}) \text{ inches} \ldots\ldots\ldots\ldots\ldots (M_1)$$

And equation (N) becomes

$$E_a + 14\frac{3.1}{4.7} + \frac{5}{8} - 7\tfrac{5}{8} = (E_a + 2\tfrac{1}{4}) \text{ inches} \ldots\ldots\ldots\ldots (N_1)$$

Therefore, we have for equal clearance $E_b - 2\frac{1}{2} = E_a + 2\frac{1}{4}$; therefore, $E_b - E_a = 4\frac{3}{4}$ inches; $E_a = 11\frac{1}{2}$ inches; and $E_b = 16\frac{1}{4}$ inches. Thus, in this case, the track should be $2\frac{1}{4}$ inches nearer the outer truss than is shown in Fig. 2, for equal clearances at the middle and ends of the span.

Now, suppose that the super-elevation of the outer rail is 3.1 inches, and that $E_a = 11\frac{1}{2}$ inches, and $E_b = 16\frac{1}{4}$ inches. If C inches is the minimum specified clearance between the trusses of a bridge on a

* The end of the car should be considered, but as it only projects $\frac{5}{16}$ inch beyond the point over truck center, it has been neglected. Eq. (M) should be $(E_b - 2 + \frac{5}{16}) = (E_b - 1\frac{11}{16})$.

straight line, the clearance in this case should be, by equations (M_1) and (N_1)

$$C + 2 \times 13\tfrac{3}{4} = (C + 27\tfrac{1}{2}) \text{ inches} \ldots\ldots\ldots\ldots\ldots\ldots (P)$$

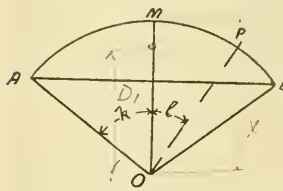
Now, in case the track is so located as to equalize the stress in the two trusses, the track must be moved toward the inner truss $6\tfrac{3}{4}$ * inches nearer than that shown in Fig. 2. In this case, $E_a = 20\tfrac{1}{2}$ inches, and $E_b = 7\tfrac{1}{4}$ inches. The clearance between the trusses must then be, by equation (N_1),

$$C + 2\,(E_a + 2\tfrac{1}{4}) = C + 2 \times 22\tfrac{3}{4} = (C + 45\tfrac{1}{2}) \text{ inches} \ldots\ldots\ldots (R)$$

This requires the width between the trusses to be just 18 inches more than is required by equation (P).

This position of the track, that will give equal stresses in the middle chords and end members of both trusses, may be found analytically as follows. Assume the equivalent uniform loading to be distributed uniformly along a curve of the same radius as the center of the track and passing through the center of gravity of the train. The center of mass of this loading, or the center of gravity of the whole train load, is situated at a point at a horizontal distance from the middle of this arc equal to $R\,\frac{S - C}{S}$, where R is the radius of the curve; S is the length of the arc, and C is the length of the chord.

This is readily proved, as follows :† Let AB be a circular arc, which is the locus of a uniform load. Let M be its middle point, and O the center of the circle. Then it is obvious from symmetry that the center of gravity of the loading must lie on the line OM. Take OM as the axis of X. Then, the ordinate of the center of gravity of an element of load at P is $D_1 = \frac{x.ds}{ds}$, where ds is the length of an elementary arc, and x is the ordinate of P, the center of gravity of the load on the elementary arc. Let l be the angle POM, and R the radius of the circle. Then



$$x = R \cos.\ l;\ ds = R.dl;\ \text{and}\ D_1 = R\,\frac{\cos.\ l\ dl}{dl}.$$

Now if the angle $BOA = 2k$, the integration must be taken from $l = -k$ to $l = k$. Therefore $D_1 = R\,\frac{2R \sin.\ k}{2Rk} = R\,\frac{C}{S}$; where C is the chord AB; and $S =$ arc AMB.

* Found by trial.

† This proof is given in Minchin's "Statics."

The distance of the center of gravity of the loading from M is therefore, $D_2 = R\left(1 - \frac{C}{S}\right) = R\frac{S-C}{S}$.

In the present case $R = 1\,910.08$ feet; $C = 187.5$ feet; $S = 187.577$ feet; and $D_2 = \left(1910.08\,\frac{187.577 - 187.5}{187.577}\right) 12 = 9.44$.

Now the circular arc through the center of gravity of the train is at a horizontal distance from the axis of the track of (60 sin. A) = (60 x 0.056) = 3.36 inches. Therefore the distance of the center of gravity of the train from the middle point of the axis of the track is $D = 9.44 + 3.36 = 12.8$ inches. The centers of gravity of the loads on the arcs AM and MB will also lie on a line parallel to the chord AB, and at 12.8 inches from the point M, because the center of gravity of the load on the whole arc AB will evidently lie on the straight line joining the centers of gravity of the partial loads AM and MB, and this line will necessarily be parallel to AB, by symmetry. If now there were no other forces acting except the vertical load, then the end reactions of both trusses and the stresses in the middle chord members of both trusses would evidently be equal, if the track were so located on the span that the middle point of the axis of the track would have a positive eccentricity of 12.8 inches with reference to the axis of the bridge.

However, the centrifugal force produces a couple whose moment is $W.\ d.\ F.\ \cos.\ A = 4.35\,W$, where W is the weight of the train. The effect of this couple is to move the line of action of W nearer the outer truss a distance of $\frac{4.35\ W}{W} = 4.35$ inches; hence in order that the resultant action of the whole loading may be at the axis of the bridge, the axis of the track must have a positive eccentricity at its middle point with reference to the axis of the bridge of $(12.8 - 4.35) = 8.45$ inches. This should be the value of E_b if the train conformed to the curvature of the track, but actually the train has the shape of a polygon, and its center of gravity is somewhat farther from M than 12.8 inches; so that E_b is a little too small. This error in the value of E_b is, however, more than compensated by the hypothesis that the eccentricity of the loading in a panel has a value equal to the mean of the eccentricities at the ends of the panel. If the value of E_b be taken at 8.5 inches, and the distance center to center of trusses be made 238 inches, the reactions will be found to be, for the inner truss 217 500 pounds, for the outer truss 215 800 pounds; and the stress on DE for the inner truss to be 380 000 pounds;

and for the outer truss to be 393 500 pounds. This shows that on account of the approximations used in determining the values of E, the value of E_b should be made somewhat less than 8.5 inches, the value found on the hypothesis of the conformity of the shape of the train to the curvature of the track. In this bridge the value of 7¼ inches for E_b appears to be about right, as is seen from the stresses in Table No. 16.

Numerical Example.—For the position of the track given by equation (R) the width center to center of trusses would be 20 feet, and we have E_1 for $b = -9.2$ inches; for $c = -0.55$ inches; for $d = +4.65$ inches; for $e = +6.37$ inches.

Equation (B) for b becomes $\frac{P}{2}\left[1 + 2\,\frac{3.36 + 9.2}{240}\right] = \frac{P}{2}\left[1 + 0.105\right]$.

" " " c " $\frac{P}{2}\left[1 + 2\,\frac{3.36 + 0.55}{240}\right] = \frac{P}{2}\left[1 + 0.033\right]$.

" " " d " $\frac{P}{2}\left[1 + 2\,\frac{3.36 - 4.65}{240}\right] = \frac{P}{2}\left[1 - 0.011\right]$.

" " " e " $\frac{P}{2}\left[1 + 2\,\frac{3.36 - 6.37}{240}\right] = \frac{P}{2}\left[1 - 0.025\right]$.

Equation (D) has values as follows:

At b... $\frac{P}{2}\left[1 - 0.105 + 0.036\right] = \frac{P}{2}\left[1 - 0.069\right]$.

At c... $\frac{P}{2}\left[1 - 0.033 + 0.036\right] = \frac{P}{2}\left[1 - 0.003\right]$.

At d... $\frac{P}{2}\left[1 + 0.011 + 0.036\right] = \frac{P}{2}\left[1 + 0.047\right]$.

At e... $\frac{P}{2}\left[1 + 0.025 + 0.036\right] = \frac{P}{2}\left[1 + 0.061\right]$.

Computation for Trusses.—The stresses in the trusses are calculated for a uniform bridge panel load of 121 600 pounds, except bB, for which the maximum truss panel load of 75 900 pounds is used.

TABLE No. 14.

PANEL LOADS FOR UNIFORM LOAD OF 121 600 POUNDS PER BRIDGE PANEL.

	b	c	d	e	f	g	h
Equation (B) for inner truss....	1.105	1.033	0.989	0.975	0.989	1.033	1.105
Equation (D) for outer truss....	0.931	1.003	1.047	1.061	1.047	1.003	0.931
Inner Panel Load.....	67 180	62 800	60 130	59 280	60 130	62 800	67 180
Outer " "	56 600	60 980	63 660	64 510	63 660	60 980	56 600

TABLE No. 15.

Reactions for Single Panel Loads.

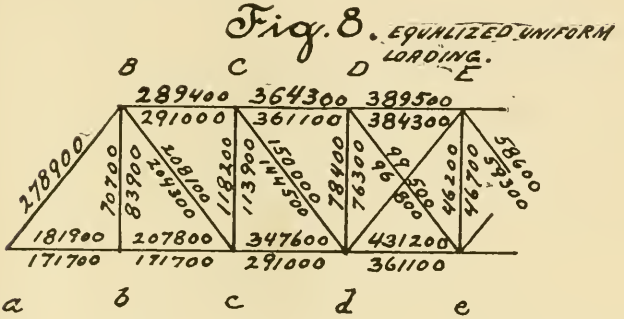
	b	*c*	*d*	*e*	*f*	*g*	*h*	Total.
Inner Truss.....	58 780	47 100	37 580	29 640	22 550	15 700	8 400	219 800
Outer Truss.....	49 530	45 740	39 790	32 260	23 870	15 240	7 080	213 500

TABLE No. 16.

Stresses in Trusses for Uniform Panel Load of 121 600 Pounds.

Member.	Truss.	Stress.		From Table No. 6 for Outer Truss.
ab	Inner	219 800 x 0.78125	171 700	*ab* 181 900
bc.....	Outer	213 500 x 0.78125 372 400	166 800	*bc* 207 800
cd	Inner	(219 800 x 2 — 67 200) x 0.78125 370 400	291 000	
BC.....	Outer	(213 500 x 2 — 56 600) x 0.78125 462 200	289 400	*cd* 347 600
de.....	Inner	(219 800 x 3 — 197 200) x 0.78125 466 300	361 100	
CD.....	Outer	(213 500 x 3 — 174 200) x 0.78125 491 900	364 300	*de* 431 200
DE.....	Inner	(219 800 x 4 — 387 300) x 0.78125 498 600	384 300	
	Outer	(213 500 x 4 — 355 400) x 0.78125	389 500	
aB.....	Inner	219 800 x 1.269	278 900	
	Outer	213 500 x 1.269 161 000	270 900	
cB.....	Inner	(219 800 — 58 800) x 1.269 164 000	204 300	
	Outer	(213 500 — 49 500) x 1.269 113 900	208 100	
dC.....	Inner	(219 800 — 105 900) x 1.269 118 200	144 500	
	Outer	(213 500 — 95 300) x 1.269 76 300	150 000	
eD.....	Inner	(219 800 — 143 500) x 1.269 78 400	96 800	
	Outer	(213 500 — 135 100) x 1.269 46 700	99 500	
fE	Inner	(219 800 — 173 100) x 1.269 46 200	59 300	
	Outer	(213 500 — 167 300) x 1.269	58 600	
bB	Inner	75 900 x 1.105	83 900	
	Outer	75 900 x 0.931	70 700	

These stresses are shown in the strain sheet, Fig. 8.



As will be seen, the stresses on the two trusses are now very nearly equal, except in the case of the lower chords and the suspenders bB; and if the actual loading were used in place of the equivalent uniform loading, the relative equality of stresses would be about the same. Evidently, the outer lower chord must, in any case, be larger than the inner chord, to provide for the additional stress due to the horizontal shear of the centrifugal force. With this exception, all the members of both trusses can be proportioned from the greatest stress on the corresponding members of both trusses, without much waste of material, if, as in this case, the track is so located as to make the average stresses on the inner and outer trusses approximately equal. This will save considerable labor in the calculation of the strain sheet, in the making of working drawings, and in the shop work, on account of the reduction in the variety of sections.

Stringers.—Now, by arranging the stringers properly with reference to the track, the maximum stresses on the inner and outer stringers may be equalized, and it will be possible to make the stringers uniform throughout. To secure this equality of stress in the stringers, equations (E) and (F) must have the value of E_2 so fixed as to make these two equations have the same value in each panel. Suppose the stringers are $7\frac{1}{2}$ feet apart, center to center, then equation (E) becomes

$$1 + \frac{3.36}{45} - \frac{E_2}{45} = 1.075 - \frac{E_2}{45} \quad \ldots\ldots\ldots\ldots\ldots\ldots (E_1)$$

And equation (F) becomes

$$1 - \frac{3.36}{45} + 0.097 + \frac{E_2}{45} = 1.022 + \frac{E_2}{45} \quad \ldots\ldots\ldots (F_1)$$

Therefore, for equality of stresses we have

$$1.022 + \frac{E_2}{45} = 1.075 - \frac{E_2}{45}, \text{ and } E_2 = 1\tfrac{3}{16} \text{ inches.}$$

If E_2 is given this value, we have equations (E_1) and (F_1), both equal to 1.049. Therefore, the end shear on each stringer becomes 57 900 x 1.049 = 60 700 pounds, and the maximum bending on each stringer, 293 000 x 1.049 = 307 400 foot-pounds.

The location of track and stringers relatively to the axis of the bridge is given in the following table:

TABLE No. 17.

ECCENTRICITY OF TRACK AND STRINGERS.

AT PANEL POINT	ECCENTRICITY FROM AXIS OF BRIDGE,			Average Eccentricity (from Center of Track) of Axis of Stringers.
	Of Center of Track.	Of Axis of Stringers.		
a	$-13\frac{3}{4} - 6\frac{3}{4} = -20\frac{1}{2} = E^a$	ab	$\left(-20\frac{1}{2} - 8\frac{5}{16}\right)\frac{1}{2} + 1\frac{3}{16} = -13\frac{3}{16}$	$1\frac{3}{16}$ inches
b	$-1\frac{9}{16} - 6\frac{3}{4} = \ldots\ldots -8\frac{5}{16}$	bc	$\left(-8\frac{5}{16} + \frac{5}{16}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots -2\frac{13}{16}$	" "
c	$7\frac{1}{16} - 6\frac{3}{4} = \ldots \ldots\ldots \frac{5}{16}$	cd	$\left(\frac{5}{16} + 5\frac{1}{2}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots\ldots + 4\frac{1}{8}$	" "
d	$12\frac{1}{4} - 6\frac{3}{4} = \ldots\ldots\ldots\ldots 5\frac{1}{2}$	de	$\left(5\frac{1}{2} + 7\frac{1}{4}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots\ldots + 7\frac{9}{16}$	" "
e	$14 - 6\frac{3}{4} = 7\frac{1}{4} = \ldots\ldots\ldots E^b$			

The equalization of stresses in the stringers can, evidently, be effected independently of the location of the track with reference to the axis of the bridge. While the advantage of uniformity of dimensions in the stringers may be thus secured; this arrangement has the disadvantage that the number of rivets attaching the stringers to the floor-beam webs is increased, since the stringers in adjacent panels are not in the same line, except at the middle floor-beam.

Floor-beams.—To find the maximum stresses in the floor-beams in this case, where the stresses in trusses and stringers are equalized, take the maximum truss panel live load of 75 900 pounds, and find the panel load on the inner and outer trusses at each point by means of equations (B) and (D). These panel loads give the shears on the floor-beams.

The bending moments on the floor-beams may be found by multiplying these panel loads by the average distance from the end of the floor-beam to the nearer stringer.

TABLE No. 18.

PANEL LOADS FOR UNIFORM LOAD OF 75 900 POUNDS PER TRUSS PANEL.

	b	*c*	*d*	*e*	*f*	*g*	*h*
Inner truss	83 860	78 400	75 060	74 000	75 060	78 400	83 860
Outer truss	70 660	76 120	79 460	80 530	79 460	76 120	70 660

The length of the floor-beams is taken at 20 feet = 240 inches.

TABLE No. 19.

BENDING MOMENTS ON FLOOR-BEAMS, FOOT-POUNDS.

BEAM.	*M* AT INNER STRINGER.	*M* AT OUTER STRINGER.
b	$83\,860\left(120-45-8\right)\frac{1}{12}=468\,200$	$70\,600\left(120-45+8\right)\frac{1}{12}=488\,800$
c	$78\,400\left(120-45+\frac{5}{8}\right)\frac{1}{12}=494\,100$	$76\,120\left(120-45-\frac{5}{8}\right)\frac{1}{12}=471\,800$
d	$75\,060\left(120-45+5\frac{7}{8}\right)\frac{1}{12}=506\,000$	$79\,460\left(120-45-5\frac{7}{8}\right)\frac{1}{12}=457\,800$
e	$74\,000\left(120-45+7\frac{9}{16}\right)\frac{1}{12}=509\,100$	$80\,530\left(120-45-7\frac{9}{16}\right)\frac{1}{12}=452\,600$

The maximum bending moments do not differ greatly, and the floor-beams may all be proportioned for a maximum shear of 83 900 pounds, and a maximum bending moment of 506 000 pounds, without involving the waste of much material. The principal items of increased cost, in case the stresses are equalized, are, for this bridge, the additional rivets required in the stringer attachments, and the increased amount of material in the floor-beams and lateral systems due to spacing the trusses 18 inches farther apart. In the bridge, as built, the floor-beams and laterals weigh about 55 000 pounds, and the increase of 18 inches in the width of the bridge will add about 10 per cent., or 5 500 pounds to the weight of metal in the floor-beams and laterals. A more important item, in many cases, would be the cost of an additional 18 inches in the width of the piers. In the bridge discussed above, the piers were already built, and were too narrow to permit the trusses to be spaced farther apart than 18½ feet.

LOCATION OF FLOOR SYSTEM.

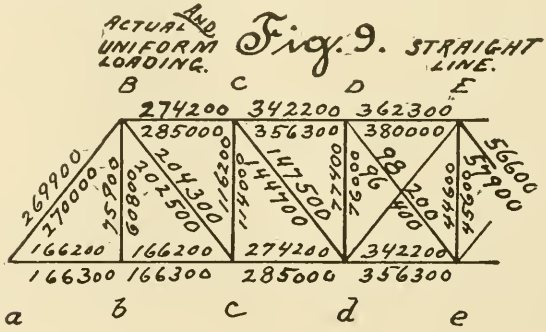
The height of the track above the plane of the lateral bracing has an important influence upon the stresses in a bridge on a curve, since any increase of this height increases the lever arm of the centrifugal force, and thus causes a change in the distribution of the loading between the

two trusses. In the bridge used in the above calculations, the floor-beams are riveted to the verticals below the lower chord pins, and the lower laterals are just above the tops of the stringers. If the floor-beams were riveted to the verticals above the lower chord pins, and the laterals were under the stringers in the plane of the lower chord centers, the value of d in equation (B) would be about 135 inches, instead of 78 inches, as in the bridge as built.

An examination of the change produced in the stresses by riveting the floor-beams to the verticals above the lower chord-pins will be interesting. Suppose the data are all the same as for the bridge discussed above, except that d is 135 inches.

Then the value of the term $\frac{2Fd \cos. A}{b}$ in equation (B) will be $\frac{2 \times 0.056 \times 135 \times 0.998}{222} = 0.068$. When d is 78 inches, the value of this term is 0.039; so that by making d equal to 135 inches, the panel loads on the outer truss, and, therefore, all the stresses on the outer truss are increased by about 3 per cent. In the case, supposed above, the shears and bending moments in the floor-beams c, d and e would evidently also be increased by 3 per cent. of the stresses that would exist for the bridge on a straight line, in addition to the stresses found for a value of 78 inches for d.

The stresses in the truss members of this bridge for the actual and uniform loading, for the track on a straight line, are given in Fig. 9; the stresses due to the actual loading are above the chords, and diagonals, and on the left of the verticals. The other stresses are due to the equivalent uniform loading of 60 800 pounds per truss panel. This strain sheet is given so as to allow a comparison to be made between the stresses for the track on a curve and on a tangent.



General Remarks.

Centrifugal Force.--Since the centrifugal force acts radially, the only point at which its line of action is perpendicular to the axis of the bridge is at the middle of the span. At every other point the centrifugal force acts at an angle with the normal to the axis of the bridge. To get the lateral centrifugal force accurately it would be necessary to find at each point the component of the centrifugal force normal to the axis of the bridge. The error resulting from neglecting to consider the inclination of the line of action of the centrifugal force to the normal to the axis of the bridge may be shown to be of no practical consequence in all ordinary cases.

The centrifugal force for a weight p is $p \frac{V^2}{32.2\,R} = p \,.\, F$, acting radially. Here $p = w \,.\, ds$; where w is the weight of the train per lineal foot, and ds is the length of an elementary arc.

Therefore the radial centrifugal force for each elementary load is $(F.\ w.\ ds)$. If the radius makes an angle l with the normal to the axis of the bridge, then the component of the centrifugal force normal to the axis of the bridge is $(F.\ w.\ \text{cos.}\ l.\ ds) = (F.\ w.\ R.\ \text{cos.}\ l.\ dl)$, since $ds = R.\ dl$.

Let k be half the angle of the curve, then the above expression integrated between $-k$ and $+k$ will give the total centrifugal force normal to the axis of the bridge, for the whole span. This is $(F.\ 2\ w.\ R.\ \text{sin.}\ k) = (F.\ w.\ C.)$, where C is the chord of the curve. The total radial centrifugal force for the whole span is found by integrating $(F.\ w.\ ds) = (F.\ w\ .\ R.\ dl)$ between $-k$ and $+k$. This gives $2\ F.\ w.\ R.\ k = F.\ w.\ S$, where S is the length of the span measured on the center line of the track. Hence the total centrifugal force acting normally to the axis of the bridge is to the total radial centrifugal force as C is to S. This ratio for 300 feet of a 10-degree curve is $\frac{296.6}{300} = 0.99$. The total error is thus seen to be about 1 per cent. of the radial centrifugal force in this extreme case. The greatest discrepancy would, of course, be at the end of the span. For the above case the ratio of the normal to the radial centrifugal force at the end of the span would be cos. 15 degrees $= 0.966$.

Where the conditions would warrant the consideration of this reduction of normal as compared with radial centrifugal force, this can be readily accomplished by multiplying the radial centrifugal force by the

cosine of the angle its line of action makes with the normal to the axis of the bridge. Thus the only change in the general formulas would be the substitution of F. cos. l for F, where l is the angle that the normal to the axis makes with the radius at the point considered. Usually it need not be considered.

Tabulation of Quantities.—The use of the general formulas in practice can be facilitated by suitable tables of the numerical values of the variables which occur in these formulas. Certain of these variables might properly be designated as variable constants, since they have one value for a given span, but are different for different spans; such quantities are b, d, f, h and k. The values of these quantities are always arbitrarily fixed for a given span; and hence it does not seem possible to construct tables that will cover the different values that may be given to these quantities. Nor does it appear that such tables would prove of much value, as the evaluation of the general formulas for given values of b, d, f, h and k is readily performed. The other variables in these formulas are the sin. A and cos. A; and the quantities F and E. The angle A is usually arbitrarily fixed, by specifying the super-elevation s of the outer rail. The sin. A is then $\frac{s}{g}$, where g is the distance between the points on the rails between which the super-elevation of the outer rail is measured. The cos. A may then be found from a table of sines and cosines. Sometimes the super-elevation s of the outer rail is made such that the component of the load parallel to the plane of the track is just equal to the component of the centrifugal force parallel to the same plane; viz., so that W. sin. $A = F$. W. cos. A, or so that $F =$ tan. A. In this case first find F and, taking this for the value of tan. A, find sin. A and cos. A from a table of circular functions. The quantities that require the most labor to evaluate are F and E, and especially the latter. These may be tabulated to advantage.

Values of F.—The value of F is given by the equation $F = \frac{V^2}{32.2\,R}$, where V is the velocity in feet per second, and R is the radius of the curve in feet. The velocity of a railway train is more commonly described as a certain number of miles per hour. Let v be the velocity in miles per hour corresponding to V feet per second; then,

$$V = \frac{5280}{3600}\,v\,;\ \text{and}\ V^2 = \frac{484}{225}\,v^2.\ \therefore\ F = \frac{v^2}{32.2\,R} \times \frac{484}{225} = \frac{v^2}{14.97\,R}\ (T).$$

On Plate XCIII the values of $\frac{v^2}{14.97}$ are laid off as ordinates, and the corresponding values of v are the abscissas. To find F for any given

velocity and radius, say 40 miles per hour for a 3-degree curve, take 40 on the axis of abscissas, read the value of the ordinate to the curve of F, in this case 107, and divide this ordinate by $R = 1910.1$. This gives

$$F = \frac{107}{1910.1} = 0.056.$$

In general, find the value of the velocity in miles per hour on the lower row of figures on the axis of abscissas; read from the plate the value of the ordinate to the curve of F at this point; and divide this ordinate by R to find the numerical value of F. The value of F is the centrifugal force per pound of load.

It will be seen that the diagram enables the values of F to be determined for any radius, and for any velocity up to 70 miles per hour. The point for a 3-degree curve and a velocity of 40 miles per hour is marked on the line of F.

Value of E.—In Fig. 10, Plate XCIII, the radius of the circle is R feet. The chord of the arc is C feet. The middle ordinate of the arc is m inches. The angle of the arc is $2\,k$.

Then we have $\sin.\ k = \frac{C}{2\,R}$ .....(V),

$m = R\,(1 - \cos.\ k) \times 12$ inches.....(W).

On Plate XCIII the values of $12\,000\,(1 - \cos.\ k)$ are laid off as ordinates, and the corresponding values of $\frac{C}{2\,R}$ are the abscissas, thus giving the curve of E.

This diagram is used as follows: For a given chord C, and radius R, find the value of $\frac{C}{2\,R}$; then look for this value of $\frac{C}{2\,R}$ on the axis of abscissas, using the *upper* row of figures; take the ordinate to the curve of E at this point, and multiply it by $\frac{R}{1\,000}$ to get the value of m in inches.

To illustrate the application of the diagram to find the values of E, take as an example the bridge of 187½ feet span, and on a 3-degree curve.

$$\text{Take } \frac{C}{2\,R} = \frac{187.5}{2 \times 1910.1} = .04908 \text{ for } C_4$$

$$\text{“ } = \frac{3}{4} \times .04908 = .03681 \text{ “ } C_3$$

$$\text{“ } = \frac{1}{2} \times .04908 = .02454 \text{ “ } C_2$$

$$\text{“ } = \frac{1}{4} \times .04908 = .01227 \text{ “ } C_1$$

Now it will be seen that the corresponding ordinates on the diagram are very small, but they can be read to the nearest half tenth without any difficulty. These ordinates are the values of 12 000 (1 — cos. k) for each corresponding value of $\frac{C}{2R}$; and the versines of the arcs corresponding to the chords c_4, c_3, c_2, c_1 may be obtained by multiplying the ordinates in the diagram by $\frac{R}{1\,000}$, which gives the value of m in equation (W). Having the value of m for each arc, it is an easy matter to find E for each panel point, as will be shown farther on. The values of m found as above are not accurate, because the value of R being so large, the values of $\frac{C}{2R}$ are small, and, as may be seen by referring to the plate, the values of m change more gradually, the smaller $\frac{C}{2R}$ becomes.

This difficulty may be avoided by taking advantage of the peculiar properties of the curve of E, as follows: This curve is approximately a parabola, as may be readily shown. Thus for a given value of the radius and chord, suppose the angle is $2\,k_1$; then $\frac{C_1}{2R} = \text{sin. } k_1$ will be the abscissa, and 12 000 (1 — cos. k_1) will be the ordinate. Now take some multiple of sin. k_1 for an abscissa, say x. sin. k_1; then the corresponding chord will be such that $\frac{C_2}{2R} = \text{sin. } k_2 = x \text{ . sin. } k_1$; and the corresponding ordinate will be 12 000 (1 — cos. k_2). But $1 - \text{cos.}^2 k_2 = \text{sin.}^2 k_2 = x^2 \text{ sin.}^2 k_1$, since the sin. k_2 equals x sin. k_1 by hypothesis.

Therefore,

$$1 - \text{cos. } k_2 = \frac{x^2 \text{ sin.}^2 k_1}{1 + \text{cos. } k_2} = \frac{x^2 (1 - \text{cos.}^2 k_1)}{1 + \text{cos. } k_2} = x^2 (1 - \text{cos. } k_1) \frac{1 + \text{cos. } k_1}{1 + \text{cos. } k_2}$$

and hence

$$12\,000 (1 - \text{cos. } k_2) = 12\,000\, x^2 (1 - \text{cos. } k_1) \frac{1 + \text{cos. } k_1}{1 + \text{cos. } k_2} \quad \ldots\ldots\ldots\ldots (e)$$

Now, the value of $\frac{1 + \text{cos. } k_1}{1 + \text{cos. } k_2}$ will be nearly unity when k_2 is small, and also when the ratio between k_1 and k_2 is small. Even when sin. k_2 is as much as 34 times as large as sin. k_1 the value of the above fraction is not much above unity, being 1.0307. The curve is actually drawn for the exact values of 12 000 (1 — cos. k), but equation (e) gives us a useful relation when we consider $\frac{1 + \text{cos. } k_1}{1 + \text{cos. } k_2} = \text{unity}$, that materially

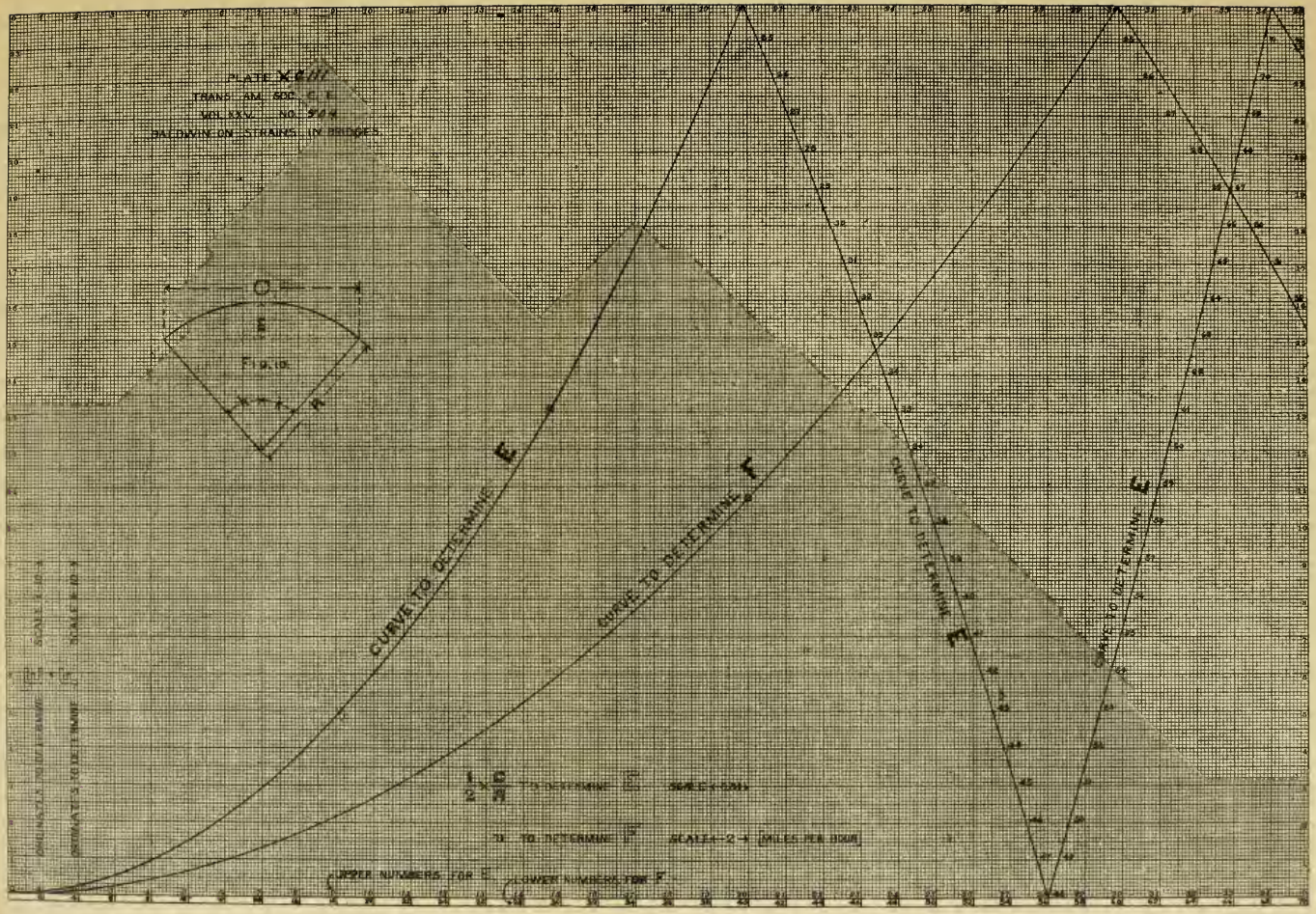


PLATE XCIII
TRANS. AM. SOC. C. E.
VOL. XXV. NO. 504
BALDWIN ON STRAINS IN BRIDGES
CURVE TO DETERMINE E
CURVE TO DETERMINE F
CURVE TO DETERMINE E
CURVE TO DETERMINE E
TO DETERMINE E
TO DETERMINE F
SCALE 1-2 MILES PER HOUR
UPPER NUMBERS FOR E
LOWER NUMBERS FOR F

increases the accuracy and range of the application of the diagram. When $\frac{1 + \text{cos. } k_1}{1 + \text{cos. } k_2}$ is made unity, we have:

$$12\,000\ (1 - \text{cos. } k_2) = 12\,000\ x^2\ (1 - \text{cos. } k_1) = A\ .\ x^2\ \ldots\ldots\ (f)$$

where $A = 12\,000\ (1 - \text{cos. } k_1)$.

Equation (f) enables us to attain minute accuracy in the cases where $\frac{C}{2R}$ is small; for we can, by using this equation, multiply our horizontal scale so as to get our ordinate at a place where the values of 12 000 (1 — cos. k) are large, and the percentage of error in reading the diagram is, therefore, small. Thus, suppose we have a value of $\frac{C_1}{2R}$, the corresponding value of 12 000 (1 — cos. k_1) being too small to be read accurately from the diagram; then multiply $\frac{C_1}{2R}$ by x, the value of x being such that the ordinate corresponding to $x\ .\ \frac{C_1}{2R} = \frac{C_2}{2R}$ will be large enough to be accurately measured. The ordinate corresponding to $\frac{C_2}{2R}$ is by equation (f), 12 000 (1 — cos. k_2) = 12 000 (1 — cos. k_1) . x^2. Therefore read the ordinate on the diagram corresponding to $x\ \frac{C_1}{2R}$, and divide it by x^2, and this latter quotient will be 12 000 (1 — cos. k_1). This latter multiplied by $\frac{R}{1\,000}$ gives m. As an illustration take the numerical example above, viz., $R = 1910.1$; $C_1 = 0.01227$; $C_2 = 0.02454$; $C_3 = 0.03681$ and $C_4 = 0.04908$. For C_1 make $x = 12$; for C_2, make $x = 6$; for C_3, make $x = 4$; and for C_4 make $x = 3$. Then

$$x\ .\ \frac{C_1}{2R} = 12 \times 0.01227 = 0.14724$$

$$x\ .\ \frac{C_2}{2R} = 6 \times 0.02454 = 0.14724$$

$$x\ .\ \frac{C_3}{2R} = 4 \times 0.03681 = 0.14724$$

$$x\ .\ \frac{C_4}{2R} = 3 \times 0.04908 = 0.14724$$

Take an abscissa of 0.1472 on the diagram, and read the ordinate corresponding to it, namely, 131. Then we have, for the versines of the arcs,

$$m_1 = \frac{131}{12^2} \times 1.9101 = 1.738 \text{ inches.}$$

$$m_2 = \frac{131}{6^2} \times 1.9101 = 6.950 \quad \text{“}$$

$$m_3 = \frac{131}{4^2} \times 1.9101 = 15.640 \quad \text{“}$$

$$m_4 = \frac{131}{3^2} \times 1.9101 = 27.800 \quad \text{“}$$

Now, by applying equations (M) and (N) we find the value of E_b for the given case to be 14 inches, say. Then the eccentricities at the panel points are,

at panel point	e				14 inches.
" " "	d	(14 — 1.74)		+ 12.26	"
" " "	c	(14 — 6.95)		+ 7.05	"
" " "	b	(14 — 15.64)		— 1.64	"
" " "	a	(14 — 27.8)		— 13.80	"

These correspond closely with the values given in Fig. 2, which latter were found analytically and by a different method. The diagram admits of the accurate determination of the versines of all curves up to an arc of 40 degrees by a simple multiplication, and may be used where it is desired to get the versines to a small fraction of an inch. No interpolation is necessary in using this diagram for F and E, but these quantities can be evaluated for any value of v less than 70 miles an hour, and for any values of R and C, provided $\frac{C}{2R}$ is less than 0.35; or, what is the same thing, provided that k is less than about $20\frac{1}{2}$ degrees.

Wind Pressure.—The general formulas and methods explained above, for finding the stresses due to centrifugal force, may, evidently, be used to find the stresses due to the side pressure of wind on the train; the values of h, d and f being measured from the center of pressure of the wind on the train, instead of from the center of gravity, and the wind pressure being substituted for centrifugal force. With these changes, and with sin. $A = 0$; $E = 0$; and cos. $A = 1$, equation (D) becomes $\frac{w \cdot d}{b}$ for a bridge on a straight line; where $w =$ the wind pressure on train per linear foot.

DISCUSSION.

William Scherzer, M. Am. Soc. C. E.—Mr. Baldwin has presented us with a very complete investigation of the problem of finding the stresses in a railroad bridge when the track is on a curve. The method of finding the position which the track should have relative to the axis of the bridge in order to equalize the stresses in the chords of inside and outside trusses, is novel, and ought to be applied wherever practicable. The local conditions will govern in each individual case as to whether it will be cheaper to increase the width between the trusses, and thereby add a certain amount of metal to the floor beams and lateral bracing, and increase the masonry, or whether it will be cheaper to adopt the minimum width between trusses required for clearance, and have a num-

ber of corresponding truss members of different sections, thereby adding to the cost in the drafting-room and shop. Where the difference in section is slight, the additional shop cost would not be of much consequence, and would be confined mainly to additional care in assembling. Corresponding pins would have the same diameter, pin plates would be of varying thicknesses, but might have the same number of rivets, thus making one templet answer for the corresponding members of both trusses.

In the case of deck truss bridges with both ends supported in the plane of the top chords, the formulas developed by Mr. Baldwin can be applied without any difficulty; but in case one end of the bridge is supported in the plane of the top chord, and the other end in the plane of the bottom chord, with inclined end posts at the latter end, the problem becomes more complicated. Perhaps Mr. Baldwin will find time to extend his investigations so as to include this case. Some years ago, the writer had occasion to calculate the stresses in a skew bridge of the latter type, with trusses of different lengths carrying a track on a 10-degree curve. It was a very difficult problem to solve mathematically, and as the time for investigation was limited, all doubtful stresses were liberally treated with impact, so as to be absolutely certain that the error was on the safe side. A great deal of the ambiguity in stresses might have been avoided if the end supports of these trusses had been kept in the plane of the top chords, making the top chords straight lines.

Viewed from a mathematical standpoint there are some assumptions in Mr. Baldwin's paper which may be open to criticism.

First.—It has been assumed that the centers of gravity of the different wheel loads have the same elevation above the tops of stringers. Is it not probable that these centers of gravity have different elevations, and that the assumption of a mean elevation would materially affect the accuracy of the results?

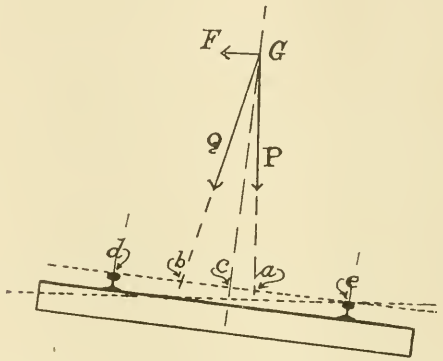
Second.—It has been assumed on page 3 that the centrifugal force acts in a plane parallel to the plane of track, which is incompatible with the assumption that the distance "f" from the center of gravity to the plane of the tops of stringers remains constant. The center of gravity describes a horizontal circle, consequently the resulting centrifugal force will be horizontal, and not inclined, as was assumed. It might be stated, however, that this error will not materially affect the numerical results in the example given.

Third.—The centrifugal forces have all been treated as normal to the axis of the bridge. In cases where the curves are very sharp it would be more correct to treat them as radial forces, and resolve them into components parallel to and normal to the axis of the bridge.

Thomas H. Johnson, M. Am. Soc. C. E.—Mr. Baldwin truly says that the complete investigation of the stresses due to curvature and its accompanying conditions has not been published. Our text books have treated that part of this subject very slightingly; possibly because the

authors regarded curves on bridges as being bad practice, and therefore not to be encouraged; or possibly they thought the elementary principles involved, so simple as not to require special elucidation. But the engineer is controlled to a certain extent by the physical conditions which surround his problem, and to which he must adapt his work. He must therefore expect to build bridges under all possible conditions of curve, grade and skew, singly and in combination; and although the principles involved, in meeting these conditions are few and simple, their application frequently becomes quite complicated; so that ready-made formulas are a grateful relief.

The work of Mr. Baldwin, as presented in this paper, is not speculative but practical, and must be productive of good. In the course of my experience I have had occasion to deal with this problem on several occasions; but approached it in a somewhat different manner from that of Mr. Baldwin. A brief account of the method pursued may not be uninteresting, and may help to throw some side light on the subject.



In the figure let G be the center of gravity of the load at a height $a\ G$ above the top of the rails, which is assumed at 5 feet. Let $d\ e$ = distance from center to center of rail heads, which for the gauge of 4 feet 9¼ inches, as used on the lines with which the writer is connected, is 4 feet 11½ inches, so that for all practical purposes the height of the center of gravity may be considered the same as the width between the centers of bearings on the rail heads. This assumption leads to a simplification, in so far as it makes the distance $c\ a$ from the center of the track to the vertical through G equal to the super-elevation of the outer rail.

[Mr. Baldwin uses the gauge of track instead of the center to center width. But the trackman in putting up his track, measures his elevation at the centers of bearing of his level-board, and the wheels, whose effect we are considering, rest on the rail heads, at the same centers. In all formulas either for super-elevation or for the effects of centrifugal force on bridges, this distance between centers should be used and not the

"gauge" in the clear between the rail heads, which has no relation whatever to the problem, either theoretical or practical.]

Let P = the rolling load.

Then F = centrifugal force = $\frac{PV^2}{32.2\ R}$ .....(1),

the notation being same as in the paper under discussion. This is the horizontal outward force applied at the rail head, and which must be transmitted as a shear through the cross-ties, stringers and floor beams to the lateral system. Its value in per cent. of P was tabulated for a speed of 50 miles per hour, and for curves from 1 to 12 degrees. This table is given herewith, and shows that for the speed assumed the centrifugal force is 2.9 per cent. of the rolling load for each degree of the curve. In the specifications for bridges—used on these lines—this is made to read 3 per cent. per degree. At 60 miles per hour the rate would become 4.2 per cent., and at 40 miles it would be 1.87 per cent.

The treatment of this horizontal force F as a panel load on the lateral system need not be further discussed here; but we will pass to the consideration of the vertical effects.

Referring to the figure, we have the vertical force P and the horizontal force F, both acting at the same time, and producing the resultant force Q, which cuts the plane of the rail heads, in the point b, and produces an unequal distribution of the load P to the two rail heads. The reactions at d and e will be respectively proportional to the segments $b\ e$ and $b\ d$, into which the distance $d\ e$ is divided by the resultant.

For the condition of train in motion the load on the outer rail only need be considered. This load will be $P^1 = \frac{P \cdot be}{de}$; be being made up of $ab - ac + \frac{1}{2}\ de$, in which $ab = \frac{F}{P}\ de$ and ac = super-elevation of outer rail.

The values of P^1 have also been tabulated in per cent. of P, and are given in the fourth column of the table. The load on the inner rail is, of course, the complement of this percentage, and the cross-tie is in the condition of a beam loaded unequally at two fixed points, from which the reactions at each of the stringers may be readily determined. That, in turn, determines two unequal loads at fixed points on the floor beam, from which the reactions at each truss are determined.

The relations of rails to stringers and to the trusses are too various in practice to permit the extension of the table to these members. The eccentricity of the track center, with reference to stringer center and truss center, present too many possible combinations to render such tabulation practicable. But when a problem of this nature is presented it is desirable to determine whether the effects of curvature are of sufficient magnitude to require special provision, without first making the laborious calculations. For this purpose the remaining columns of the table have been prepared, based on all the conditions involved except

the eccentricity of the track center. This eccentricity will be outward at the center of the span and inward at the ends of the span, tending to equalize them, hence this part of the table may be considered as fairly representing the effect of curvature upon the outer truss as a whole, but must not be used to find the effect at each panel point in detail. It serves as a valuable aid to judgment, and shows whether or not the more laborious calculation must be undertaken or may be disregarded; but it should not take the place of such calculations, except, perhaps, in the case of stringers and plate girders spaced 6 feet 6 inches center to center, in which case the column headed with that width is close enough for practice.

For the condition of a train at rest, there being no horizontal component, the reactions at the two rails are determined by the sections *ad* and *ae*, into which the width *de* is divided by the vertical P, which cuts the plane of the rail heads at the point *a*, distant from *c* by an amount *ac*, equal to the super-elevation. In this case the inner rail receives the larger share, its load being represented by

$$P'' = \frac{P \,.\, ad}{de}$$

which is also given in per cent. of P in the fifth column of the table.

TABLE

SHOWING THE HORIZONTAL AND VERTICAL EFFECTS OF CENTRIFUGAL FORCE ON BRIDGES.

Velocity = 50 miles per hour.

Curve.	Radius.	Centrifugal Force.	% of P on		Excess Load on outer Girder expressed in % of ½ P, the normal load.						
			Outer Rail.	Inner Rail.	Widths between Girders.						
Degree.	Feet.	% of P.	Motion.	Rest.	6.5	10	11	12	13	14	15
1	5 730	2.9	51.2	51.6	2.0	1.4	1.2	1.1	1.0	0.9	0.8
2	2 865	5.8	51.3	51.6	6.4	4.2	3.6	3.4	3.0	2.8	2.6
3	1 910	8.7	56.3	52.5	9.6	6.4	5.4	5.0	4.6	4.2	4.0
4	1 433	11.6	58.4	53.3	12.8	8.4	7.2	6.6	6.2	5.8	5.4
5	1 146	14.6	60.4	54.2	16.0	10.6	9.0	8.4	7.8	7.2	6.6
6	955	17.5	62.5	55.0	19.2	12.6	11.0	10.0	9.2	8.6	8.0
7	819	20.4	64.6	55.8	22.4	14.8	12.8	11.6	10.8	10.0	9.4
8	717	23.3	66.7	56.6	25.6	16.8	14.6	13.4	12.4	11.4	10.6
9	637	26.2	68.8	57.5	28.8	19.0	16.4	15.0	13.8	12.8	12.0
10	574	29.1	70.8	58.3	32.0	21.0	18.2	16.6	15.4	14.2	13.4
11	522	32.0	72.9	59.2	35.2	23.2	20.0	18.4	17.0	15.8	14.6
12	478	34.9	75.0	60.0	38.4	25.2	21.8	20.0	18.4	17.2	16.0

E. THACHER, M. Am. Soc. C. E.—I have read Mr. Baldwin's paper with interest and he is entitled to much credit for the thorough and able manner in which he has handled the subject. The formulas given are not difficult of application and will be of service to the profession.

In my own practice in problems of this kind, I have not considered it necessary to go into any great niceties of calculation, but have been satisfied to use such rapid approximate methods as will give results as close to the truth as the data on which they are based, but with formulas ready made, greater exactness can be practiced without great sacrifice of time. To use the actual wheel loads complicates the problem and multiplies the work, and I do not consider it necessary or desirable. To substitute for the wheel loads a uniform load such as will give the maximum shear at the ends of the bridge, gives in the web members essentially the same stresses as from the wheel loads, but slightly in excess toward the center and on the counters, where an excess is usually provided in any event. In the chords at or near the center, the uniform load gives quite a materially greater result, but if the train is considered to precede as well as follow the engines this difference disappears, and the uniform and wheel loads give almost identically the same results; and as this method of loading is not impossible, nor in fact very uncommon, it is questionable if the use of the uniform load is not the best practice.

It appears to be considered by some engineers that the use of a uniform load of any kind is but a rough approximation, but this must arise from a misunderstanding or wrong application of it. For some years past I have been accustomed to use wheel loads in calculation, and to check by uniform loads, which allows of a direct comparison of stresses by the two methods in every estimate. I use the wheel loads not because they are considered better than a uniform load, but simply because the railway companies are satisfied with so doing, and it gives a little cheaper bridge. The use of uniform loads in the calculation of track stringers, floor beams, long suspenders and floor beam hangers, simply allows the use of previously prepared tables; and as the results are the same as from the wheel loads, except slight differences due to interpolation, there is no choice except in time between the two methods. For track stringers the length of span becomes the length of a panel, and the uniform load used gives the same maximum moment as the wheel loads. For floor beams, suspenders and hangers, the length of span becomes the length of two panels.

Equation M for equalizing clearance appears to be taken for a point over the center of the truck, whereas the corner of the car should be taken. Taking a Pullman car 49 feet between centers of trucks and 70 feet in total length, the ordinate for 49 feet $= 1\frac{3}{4}$ inches, and the ordinate at the end of the car $= 2\frac{1}{4}$ inches. $\therefore E_b + \frac{1}{4}'' = Ea + \frac{7}{8}''$ or $E_b + Ea = \frac{5}{8}''$ instead of $1\frac{3}{4}$ inches as given.

I cannot quite agree with the author that the height of the track above the plane of the lateral bracing has an important influence upon the stresses in a bridge on a curve. It appears to me that the lever arm of the couple should be measured from the center of gravity of the

train to the center of the bottom chords and not to the center of the lateral system. The laterals, if not placed at the center of the chords, where they belong, exert an excentric stress on the posts, which must be taken care of. The end reactions are a summation of loads at the panel points which govern the stresses in the bridge, and would appear to me to be independent of the position of the lateral bracing. Again, suppose it was possible to place the lateral system at the center of gravity of the train, then the moment of the couple becomes zero, and the centrifugal force has no effect on the stresses, which evidently is incorrect, as the reactions remain unchanged. If the above views are correct, the plane of lateral bracing in Fig. 3, should be marked plane of bottom chords and the notation be made to agree, the formulas remaining unchanged.

HARRY B. SEAMAN, M. Am. Soc. C. E.—The author is in error, in computing a vertical component of the centrifugal force due to the elevation of the outer rail. The centrifugal force acts at the center of gravity of the moving body in the direction of the radius of the curve of motion, and is independent of the shape of the body or its position due to super-elevation.

The action of the centrifugal force noted, as increasing the strains in the outer truss, is interesting and would be even more marked in the case of deck bridges with trusses closer together. With the height of the center of gravity, the distance between trusses, the speed and curvature, each constant, the effect of the centrifugal force on the outer truss is a fixed ratio of the moving load.

WARD BALDWIN, M. Am. Soc. C. E. (in reply to the Discussion).—Mr. Scherzer criticises the assumption of a uniform height of the centers of gravity of the different wheel loads. It is true that this is only a rough approximation, but the variations in the heights of these centers for a maximum train load, such as is usually specified for bridge loadings, would not, in my opinion, be great. I am not aware of the existence of any exact data on this point, but here, as is not unusual, the engineer must exercise his judgment, so as to cover probable contingencies, without regard to mathematical precision. For a train of loaded freight cars, equivalent to a uniform load of from 3 000 to 4 000 pounds per lineal foot, hauled by consolidation engines with from 35 000 to 40 000 pounds on a driving axle, the center of gravity of any part of the train would, in my judgment, be between 5 and 6 feet above the tops of the rails. I think 5 feet full small for this height, and believe 5.5 feet would be nearer the truth. As it was my purpose in part to show that the stresses due to centrifugal forces were of sufficient magnitude to deserve attention, I purposely took a minimum value for h, so as to avoid the appearance of exaggeration.

The second criticism offered by Mr. Scherzer is accepted as a practical improvement, and I have modified my paper to accord with the assumption that the center of gravity describes a horizontal circle, and that the

centrifugal force therefore acts horizontally. This is not always the case, however. In fact, in the case of the bridge discussed, the elevation of the outer rail was secured by giving the ties a uniform inclination to the horizon, so that the tops of the cross-ties are all in the same plane, inclined at the angle "A" to the horizon; and the center line of the track at the middle of the span is higher than the center line of the track at the ends of the span by (m. tan. $A = 1.6$ inches), where m is the middle ordinate of the track on the span, or $27\frac{3}{4}$ inches. Thus f is not a constant, but may be assumed so without material error. So, also, while the center of the track is really in the form of an ellipse, inclined to the horizon, still it can be considered a horizontal circle, as Mr. Scherzer suggests, with a close approximation to the truth; and with the advantage of getting simpler equations for the general formula, although not always mathematically correct.

The third criticism offered by Mr. Scherzer suggests a point which I have discussed in the body of my paper. I there show that the normal differs from the radial centrifugal force by only 1 per cent. (of the radial centrifugal force) in the case of a 10-degree curve 300 feet long; and so conclude that the resolution of the centrifugal force is usually not necessary.

Mr. Thacher is correct in saying that the corner of the car should be taken in equation (M); but he has used a Pullman car to figure clearance. I have used a freight car 34 feet long and 28 feet on truck centers, because I have considered it of more importance to so locate the track as to secure equal side clearances for freight than for passenger cars. The ordinate for 28 feet is $\frac{5}{8}$ inch and the ordinate for 34 feet is $1\frac{5}{16}$ inch; so that if the truck of the car is at the middle of the span, the value of equation (M) for the end of the car at the floor level would be $(E_b - 2 + \frac{5}{16}) = (E_b - 1\frac{11}{16})$ inches. This gives for equal clearances for the freight car $E_b - 1\frac{11}{16} = E_a - \frac{1}{4}$ $\therefore$ $E_b - E_a = 1\frac{7}{16}$ inches, instead of $1\frac{3}{4}$ inches, as originally given. In the case of the Pullman car, the $2\frac{1}{4}$-inch ordinate at the end of the car is not the projection of the end of the car beyond the truck, but is the middle ordinate of an arc of 70 feet. The projection of the corner of the car beyond the edge of the car at the truck center is $2\frac{1}{4} - 1\frac{3}{4}$ inches $= \frac{1}{2}$ inch. Therefore, for a Pullman car, equations (M) and (N) give $E_b - 1\frac{1}{2} = E_a + \frac{7}{8}$ inches or $E_b - E_a = 2\frac{3}{8}$ inches. The track should, however, in my opinion, be adjusted to give equal clearances for freight cars.

Mr. Thacher has evidently not understood my discussion of the effect of changing the floor system from a position below the bottom chord to a position above the bottom chord. I do not discuss the effect of moving the lateral bracing out of the plane of the chord-pin centers. The two cases were, first, the floor-beams attached below the bottom chord; second, the floor-beams attached to posts above the bottom chord. In both cases the laterals were supposed to be in the plane of the bottom chord-pin centers. I have modified my language in this part of my

paper so as to make this point clearer. Of course, the laterals ought to always be in the plane of the lower chord-pin centers. If, however, it were possible to place the lateral system at the center of gravity of the train, as Mr. Thacher suggests, then, as he says, the moment of the couple becomes zero ; but the horizontal action of the centrifugal force is not annulled, and it would produce its full effect upon the laterals, and put a cross strain on the posts unless there were a longitudinal chord at the same level as the attachments of the laterals to the posts. The horizontal reactions due to the centrifugal force are always the same, but the vertical reactions depend entirely upon the relative positions of the center of gravity of the load and the plane of the lateral bracing, which latter ought to always coincide with the plane of the bottom chord-pin centers in a through span.

If the center of gravity is above the lateral bracing the vertical reaction of the outer truss is increased; if the center of gravity is in the plane of the lateral bracing, the vertical reaction of neither truss is affected by the centrifugal force ; if the center of gravity is below the lateral bracing, the vertical reaction of the inner truss is increased. I think Mr. Thacher will agree with my views when he understands what they are.

The measurement of the super-elevation of the outer rail between the gauge lines, to which Mr. Johnson takes exception, gives the same value for A that would be obtained if the value of g were made the distance from center to center of rail heads ; for the ratio $\frac{s}{g}$ is the quantity that appears in the equations, and evidently s and g increase in the same proportion as long as A is constant. While it may be in accordance with practice to refer s and g to the centers of the rail heads, the numerical results will not be affected by such change in the values of s and g, since $\frac{s}{g}$ remains the same.

Mr. Seaman is mistaken in supposing that I consider the centrifugal force to act in any other way than radially. The center of gravity of the train does not always move in a horizontal circle, as Mr. Seaman assumes. This is only true when the axis of the stringers has the same curve as the track. In the bridge, used as an example in this paper, the tops of the cross-ties are in a plane inclined at an angle A to the horizontal; the center line of the track is laid out with a horizontal radius, and therefore lies on the surface of a vertical cylinder whose radius is the nominal radius of the track. The plane of the cross-ties, being inclined to the axis of the cylinder, cuts the surface of the cylinder in an ellipse; and therefore the actual path of the center of gravity of the train is an ellipse in a plane parallel to the plane of the cross-ties; the radius of the path of the center of gravity at any point is the radius of curvature of this ellipse at this point; and the radial centrifugal force is parallel to the plane of the cross-ties, just as I took it to be. Practically, when A is small, this refinement can be ignored, and the radius be assumed to be horizontal.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

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510.
(Vol. XXV.—November, 1891.)

"DIMENSION STONE QUARRYING.—THE BLASTING PROCESS."

By WILLIAM L. SAUNDERS, M. Am. Soc. C. E.

WITH DISCUSSION.

There are doubtless many persons, some of them, perhaps, not without a knowledge of quarrying, who have read extracts from the Census Reports, and have noted frequent reference to a system of blasting known as the Knox system. This system is a recent invention, no mention was made of it in the tenth census, and no description has yet been given of it in any publications on quarrying. The first work done by this method was in 1885, and at the close of that year two quarries had adopted it. In 1886 it was used in twenty quarries. In 1887 in forty-four, in 1888 in upward of one hundred, and at the present time about three hundred quarries have adopted it. Its purpose is to release dimension stone from its place in the bed, by so directing an explosive force that it is made to cleave the rock in a prescribed line and without injury. The system is also used for breaking up detached blocks of stone into smaller sizes.

Quarrymen have, ever since the introduction of blasting, tried to direct the blast so as to save stock. Holes drilled by hand are seldom round. The shape of the bit and the irregular rotation while drilling, usually produce a hole with a triangular section, the walls of the hole taking the shape shown in Fig. 1. It was observed, many years ago, that when a blast was fired in a hand-drilled hole, the rock usually broke in three directions radiating from the points of the triangle in the hole. This led quarrymen to look for a means by which the hole might be shaped in accordance with a prescribed direction of cleavage.

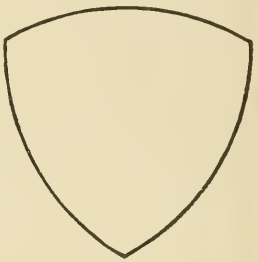


FIG. 1.

The oldest sandstone quarries in America are those at Portland, Conn. It was from these quarries that great quantities of brownstone were shipped for buildings in New York. The typical "brownstone front" is all built of Portland stone. As the Portland quarries were carried to great depths the thickness of bed increased, as it usually does in quarries. With beds from 10 to 20 feet deep, all of solid and valuable brownstone, it became a matter of importance that some device should be applied which would shear the stone from its bed without loss of stock and without the necessity of making artificial beds at short distances. A system was adopted and used successfully for a number of years which comprised the drilling of deep holes from 10 to 12 inches in diameter, and charging them with explosives placed in a canister of peculiar shape. The drilling of this hole is so interesting as to warrant a passing notice. The system was similar to that followed with the old-fashioned drop drill. The weight of the bit was the force which struck the blow, and this bit was simply raised or lowered by a crank turned by two men at the wheel. The bit resembled a broad axe in shape, in that it was extremely broad, tapering to a sharp point and convex along the edge.

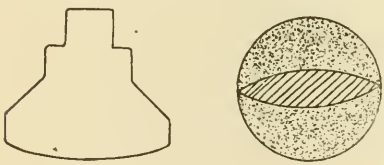


FIG. 2.

Fig. 2 illustrates in section, one of the Portland drills, and a drill-hole with the canister containing the explosive in place. This canister was lune-shaped, that is, its section was bounded by two minor segments of a circle. The canister was made of two curved pieces of sheet tin with soldered edges, cloth or paper being used at the ends. It was

surrounded with sand or earth, so that the effect of the blast was practically the same as though the hole were drilled in the shape of the canister. In other words, the old Portland system was to drill a large, round hole, put in a canister, and then fill up a good part of the hole. Were it possible to drill the hole in the shape of the canister, it would obviously save a good deal of work which had to be undone. The Portland system was, therefore, an extravagant one, but the results accomplished were such as to fully warrant its use. Straight and true breaks were made, following the line of the longer axis of the canister section. See Fig. 3.

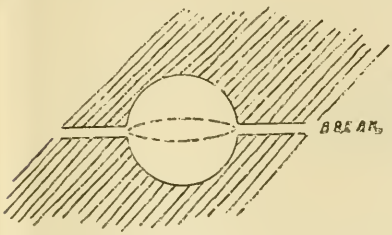


FIG. 3.

It was found that with the old Portland canister two breaks might be made at right angles by a single blast, when using a canister shaped like a square prism. In some of the larger blasts, where blocks weighing in the neighborhood of 2 000 tons were sheared on the bed, two holes as deep as 20 feet were drilled close together, the core between the holes was then chipped out and large canisters measuring 2 feet across from edge to edge were used.

Another of the older systems of blasting is that known as Lewising, Fig. 4. A Lewis hole is made by drilling two or three holes close together and parallel with each other, the partitions between the holes being broken down by using what is known as a broach. Thus a wide hole or groove is formed in which powder is inserted, either by ramming it directly in the hole, or by putting it in a canister, shaped somewhat like the Lewis hole trench. A complex Lewis hole is the combination of three drill holes, while a compound Lewis hole contains four holes. Lewising is confined almost entirely to granite. In some cases a series of Lewis holes is put in along the bench at distances of 10 and 25 feet apart, or even greater, each Lewis hole being situated equidistant from the face of the bench. The holes are blasted simultaneously by the electric battery.

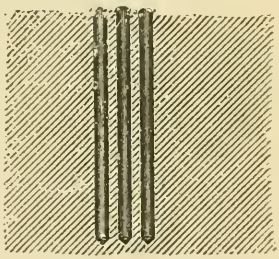


FIG. 4.

Another system used to a limited extent, and by no means to be commended, is one involving the use of inverted plugs and feathers. The hole, which in this case is an ordinary one, is first charged with powder, then the plugs and feathers are inserted as a sort of tamping; the effect

FIG. 5.

FIG. 7.

FIG. 6.

of the blast being to drive the plugs between the feathers and to split the rock by the usual plug and feather process. This is rather a violent way of using this process, and it frequently results in irregular breaks and in damage to the rock at the top of the hole.

It is thus seen that the "state of the art" has been progressive, though it was imperfect. Mr. Sperr, in his reference to this subject made in the report of the Tenth Census, says: "The influence of the shape of the drill-hole upon the effects of the blast does not seem to be generally known, and a great waste of material necessarily follows." This was written but a few years before the introduction of the new system, and it is doubtless true that attention was thus widely directed to the conspicuous waste due to a lack of knowledge of the influence of the shape of a drill-hole on the effect of a blast. The system developed by Mr. Knox practically does all and more than was done by the old Portland system, and it does it at far less expense. It can best be described by illustrations.

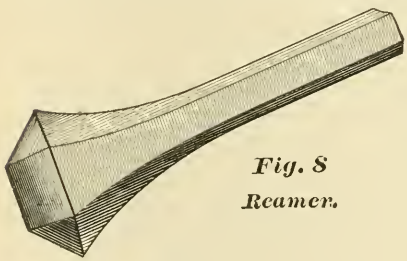


Fig. 8
Reamer.

Fig. 5 is a round hole drilled either by hand or otherwise, preferably otherwise, because an important point is to get it round. Fig. 6 is the improved form of hole, and this is made by inserting a reamer, Fig. 7, into the hole in the line of the proposed fracture, thus cutting two V-shaped grooves into the walls of the hole. The reamers are further shown in Fig. 8. The blacksmith tools for dressing the reamers are shown in Fig. 9. The usual method of charging and tamping a hole in using the new system is shown in Fig 10. The charge of powder is shown at *C*, the air-space at *B*, and the tamping at *A*.

Fig. 11 is a special hole for use in thin beds of rock. The charge of powder is shown at *D*, the rod to sustain tamping at *B*, air-space at *CC*, and tamping at *A*.

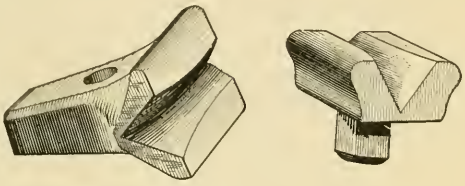


Fig. 9.

Let us assume that we have a bluestone quarry in which we may illustrate the simplest application of the new system. The sheet of stone which we wish to shear from place has a bed running horizontally at a depth of, say, 10 feet. One face is in front, and a natural seam divides the bed at each end at the walls of the quarry. We now have a block of stone, say 50 feet long, with all its faces free, except one—that opposite and corresponding with the bench. One or more of the specially formed

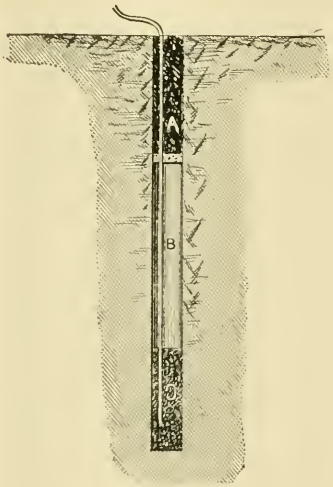


Fig. 10.

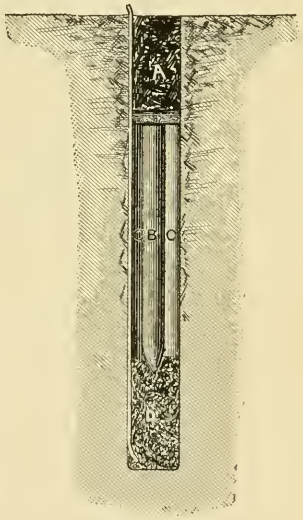


Fig. 11.

holes are put in of such depth and distance apart, and from the bench, as may be regulated by the thickness, strength and character of the rock. No man is so good a judge of this as the quarry foreman who has used and studied the effect of this system in his quarry. Great care should be taken to drill the holes round and in a straight line. In sandstone of medium hardness these holes may be situated 10, 12, or 15 feet apart.

If the bed is a tight one, that is where it is not entirely free at the bottom, the hole should be run entirely through the sheet and to the bed; but with an open free bed, holes of less depth will suffice.

The reamer should now be used and driven by hand. Several devices have been applied to rock drills for reaming the hole by machinery while drilling; that is, efforts have been made to combine the drill and the reamer. Such efforts have met with only partial success. The perfect alignment of the reamer is so important that where power is used this point is apt to be neglected. It is also a well known fact that the process of reaming by hand is not a difficult or a slow one. The drilling of the hole requires the greatest amount of work. After this has been done it is a simple matter to cut the V-shaped grooves. The reamer should be applied at the center of the hole, that is, the grooves should be cut on the axis or full diameter of the hole. The gauge of the reamer should be at least 1½ times the diameter of the hole. While driving the reamer great care should be taken that it does not twist, as the break may thereby be deflected; and the reaming must be done also to the full depth of the hole.

The hole is now ready for charging. The powder should be a low grade of explosive. Dynamite is not suitable, and Black powder, Judson powder, or other explosives which act slowly are preferable. No definite rule can be laid down as to the amount of powder to be used, but it is well to bear in mind that it should be as small as possible. As a matter of fact, very little powder is required in most rocks. Hard and fine grained stone requires less powder than soft stone. This is based on the same principle of philosophy that granite will split by the plug and feather process with holes of less diameter and depth and with less expenditure of force than sandstone. Mr. Knox tells of a case which came under his observation, where a block of granite "more than 400 tons weight, split clear in two with 13 ounces of FF powder." He compares this with a block of sandstone of less than 100 tons weight "barely started with 2½ pounds of the same grade of powder, and requiring a second shot to remove it."

It is obvious that enough powder must be inserted in the hole to produce a force sufficient to move the entire mass of rock on its bed. In some kinds of stone, notably sandstone, the material is so soft that it will break when acted upon by the force necessary to shear the block. In cases of this kind a number of holes should be drilled and fired

simultaneously by the electric battery. In such work it is usual to put in the holes only 4 or 5 feet apart. The powder must, of course, be provided with a fuse or preferably a fulminating cap. It is well to insert the cap at or near the bottom of the cartridge, as shown in Figs. 10 and 11.

After the charge the usual thing to do is to insert tamping, but in the improved form of hole the tamping should not be put directly upon the powder, but an air space should be left, as shown at *B*, Fig. 10. The best way to tamp, leaving an air space, is first to insert a wad which may be of oakum, hay, grass, paper or other similar material. The tamping should be placed from 6 to 12 inches below the mouth of the hole. In some kinds of stone a less distance will suffice, and it is well to bear in mind that as much air space as practicable should intervene between the explosive and the tamping. Care should be observed in tamping not to destroy the wires which connect with the explosive, but the tamping should be made secure, so that it will not blow out. The hole is now ready for blasting. If several holes are used on a line they should be connected in series, Fig. 12, and blasted simultaneously by electricity. The effect of the blast is to make a vertical seam connecting the holes, and the entire mass of rock is sheared several inches or more.

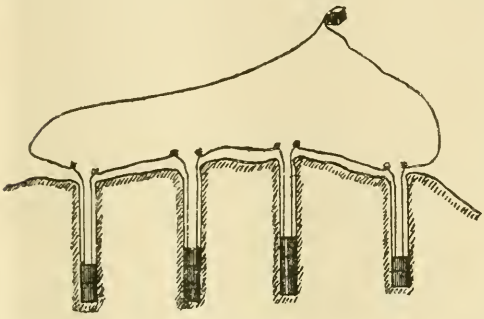


FIG. 12.

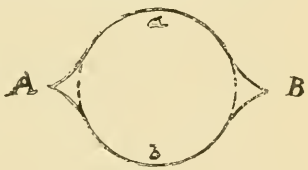


FIG. 13.

The philosophy of this new method of blasting is simple, though a matter of some dispute. The following explanation has been given. See Fig. 13.

"The two surfaces, *a* and *b*, being of equal area, must receive an equal amount of the force generated by the conversion of the explosive into gas. These surfaces being smooth and presenting no angle between the points *A* and *B*, furnish no starting point for a fracture, but at these points the lines meet at a sharp angle including between

them a wedge-shaped space. The gas acting equally in all directions from the center is forced into the two opposite wedge-shaped spaces, and the impact being instantaneous, the effect is precisely similar to that of two solid wedges driven from the center by a force equally prompt and energetic. All rocks possess the property of elasticity in a greater or less degree, and this principle being excited to the point of rupture at the points *A* and *B*, the gas enters the crack and the rock is split in a straight line, simply because under the circumstances it cannot split in any other way."

Another theory has been stated as follows: "A round hole forms on all sides a perfect arch, and, if the rock be sound, it is equally strong in all directions. The making of the grooves at opposite sides of the hole breaks the arch at these sides, thus producing two weak sides and two strong sides at right angles with each other. Force being applied within the hole for the purpose of breaking the rock, naturally exerts itself in the lines of weakness which have been produced by destroying the arches at *A* and *B*, and they, being exactly opposite to each other, the result is that the rock is fractured in a straight line; the gas generated by the explosion acting in these lines in the same manner as a line of wedges would, if applied from the outer side of a rock, with this difference, viz.: Wedges applied from the outer side of a rock are driven inward toward the point of greatest resistance in the rock, whilst the gas being confined within the rock at its strongest part and operating toward the outside, or weaker part, will naturally take that direction which will most quickly relieve the pressure, and that is a direct line to the surface."

The effect of the new system being practically the same as that of the old Portland system, or that of Lewising, it is natural that we should look into these systems in our efforts to explain why the rock breaks in a prescribed line. Good and true breaks have been made by Lewising, yet there is no V-shaped groove. Equally clear and efficient is the record of the old Portland canister, yet here too there are no V-shaped grooves. It might be argued that the Portland canister being imbedded in sand or other non-elastic material, forms a "wedge-shaped space," and here too, "The gas is forced into the two opposite 'wedge-shaped spaces,' and the impact being instantaneous, the effect is precisely similar to that of two solid wedges driven from the center by a force equally prompt and energetic."

But we are met by the evidence of the Lewis-hole, where there is no "wedge-shaped space." While it is doubtless true that the "wedge-

shaped space" is an influence which assists the break, and that the breaking of the arch of the hole by the groove renders equally great assistance in that it produces a weak point to start the break, yet the main cause in the new form of blast which acts to direct the break is, that the lines of force are exerted against the surfaces of the hole, *A b B* and *A a B*, to a greater extent than upon the surfaces *a A b* and *a B b*. In other words, there is a greater area of pressure acting toward *a* and *b*, and this naturally tends to produce a separation at *A* and *B*. The tension is precisely the same as that produced by a line of plugs and feathers, or by a series of wedges driven in a trench.

Let us assume that an effort was made to split by the new form of holes without an explosion, but by using hydraulic or other pressure within the holes. It is obvious that were we able to get the pressure high enough, the break would be made in the same way as though it were blasted, and the very purpose of the air cushion in the new system is to prevent the shock of the blast from having a bad effect upon the rock, and to confine the gas and air at a high pressure in the hole. The suddenness of the explosion is relieved by an expansion and compression of the air cushion, and it is the recoil of the energy after such compression which exercises the greatest force in breaking the rock. Notwithstanding the cushion there is some shock, and this so far assists the break as to enable the operator to use but little explosive. Were the force exerted through hydraulic pressure, it would be advisable to produce a shock, just as a quarryman will strike a block of granite with a heavy sledge in order to start the break, while his plugs and feathers are under strain.

The new form of hole is, therefore, almost identical in principle with the old Portland canister, except that it has the great advantage of the shaped groove in the rock, which serves as a starting point for the break. It is also more economical than the Portland canister in that it requires less drilling and the waste of stone is less. It is, therefore, not only more economical than any other system of blasting, but it is more certain, and in this respect it is vastly superior to any other blasting system, because stone is valuable, and anything which adds to the certainty of the break also adds to the profit of the quarryman.

It is doubtless true that, notwithstanding the greater area of pressure in the new form of hole, the break would not invariably follow the prescribed line but for the V-shaped groove which virtually starts it. A

bolt, when strained, will break in the thread whether this be the smallest section or not, because the thread is a starting point for the break. A rod of glass is broken with a slight jar provided a groove has been filed in its surface. Numerous other instances might be cited to prove the value of the groove. Elasticity in rock is a pronounced feature, which varies to a greater or less extent; but it is always more or less present. A sandstone has recently been found which possesses the property of elasticity to such an extent that it may be bent like a thin piece of steel. When a blast is made in the new form of hole the stone is under high tension, and being elastic it will naturally pull apart on such lines of weakness as grooves, especially when they are made as is usually the case in this system, in a direction at right angles with the lines of least resistance.

Our previous illustration of a break by the new system was its simplest and best application. An identical case would be one where a large and loose block of stone was split up into smaller ones by one or more of this form of holes. But those who use this system do not confine it to such cases alone. Horizontal holes are frequently put in and artificial beds made by "lofting." In such cases where the rock has a "rift" parallel with the bed, one hole about half way through is sufficient for a block about 15 feet square, but in "liver" rock the holes must be drilled nearly through the block and the size of the block first reduced.

A more difficult application of the system, and one requiring greater care in its successful use, is where the block of stone is situated, as in the case hereinbefore cited, except that both ends are not free, one of them being solidly fixed in the quarry wall. A simple illustration of a case of this kind is a stone step on a stair-way which leads up and along a wall. Each step has one end fixed to the wall and the other free. Each step is also free on top, on the bottom and on the face, but fixed at the back. We now put one of the new form of holes in the corner at the junction of the step and the wall. The shape of the hole is as shown in Fig. 14.

It is here seen that the grooves are at right angles with each other, and the block of stone is sheared by a break made opposite and parallel with the bench, as in the previous case, and an additional break made at right angles with the bench and at the fixed end of the block. Sometimes a corner break is made by putting in two of the regular V-shaped holes in the lines of the proposed break and without the use of the

corner hole. A useful application of this system is in splitting up large masses of loose stone. For this purpose the V-shaped grooves are

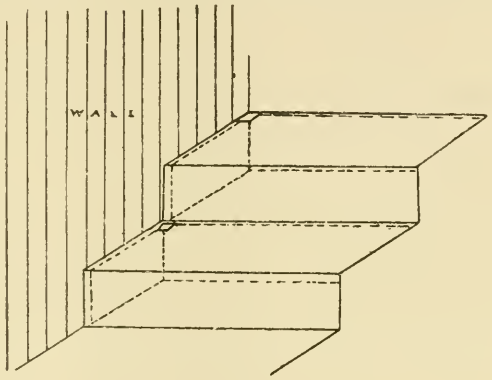


FIG. 14.

sometimes cut in four positions and breaks are made in four directions radiating from the center of the hole as shown in Fig. 15. In this way a block is divided into four rectangular pieces.

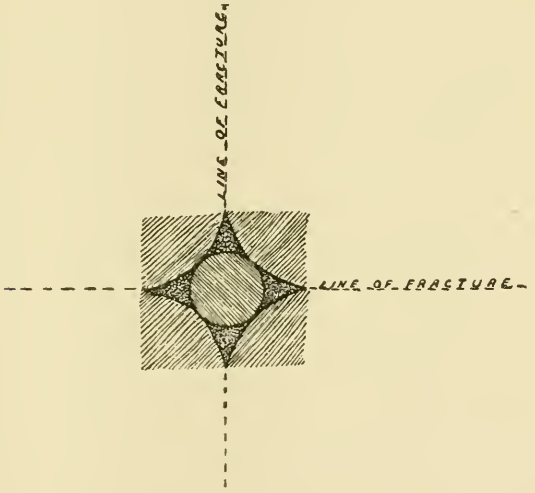


FIG. 15.

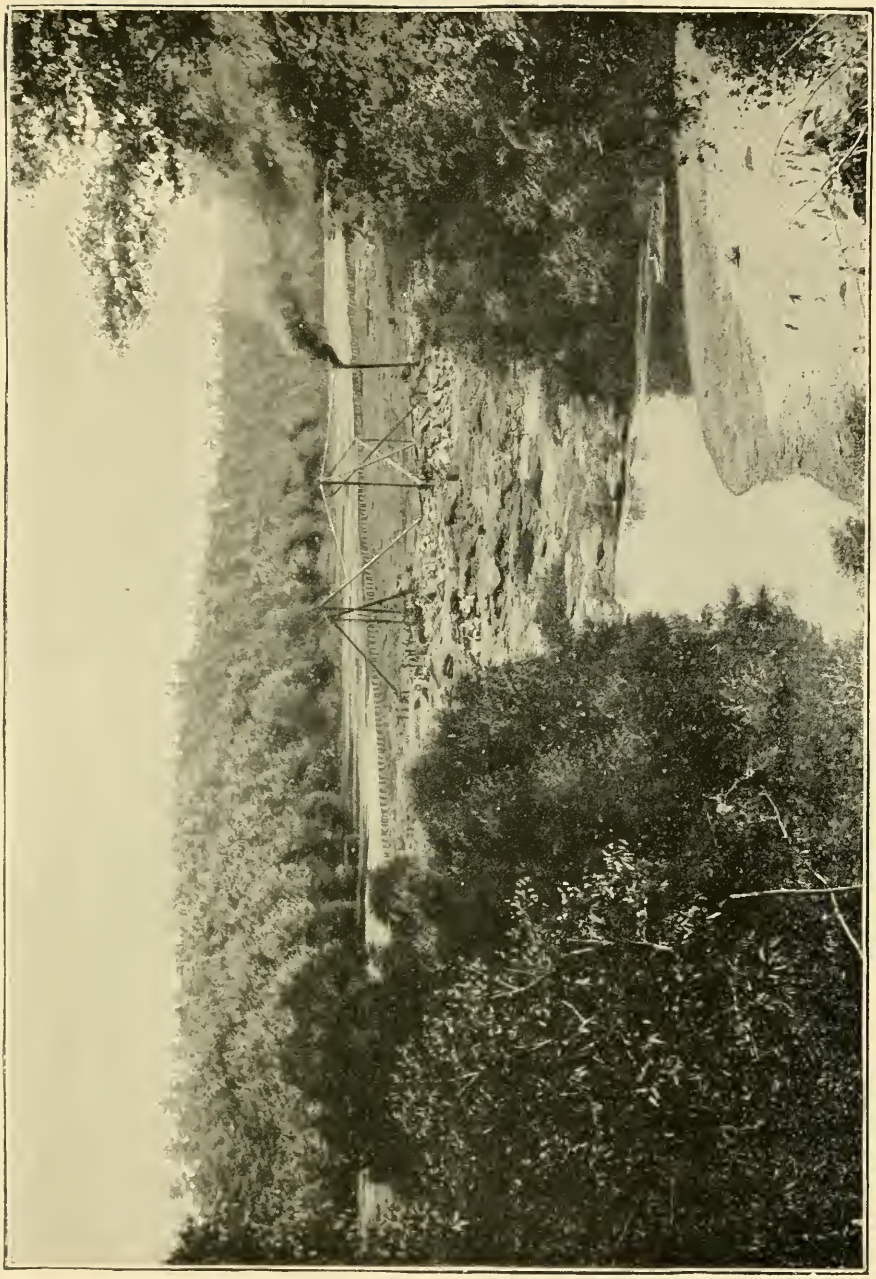
Though the new system is especially adapted to the removal of heavy masses of rock, yet it has been applied with success in cases where several light beds overlie each other. In one such instance ten sheets, measuring in all only 6 feet, were broken by a blast, but in cases

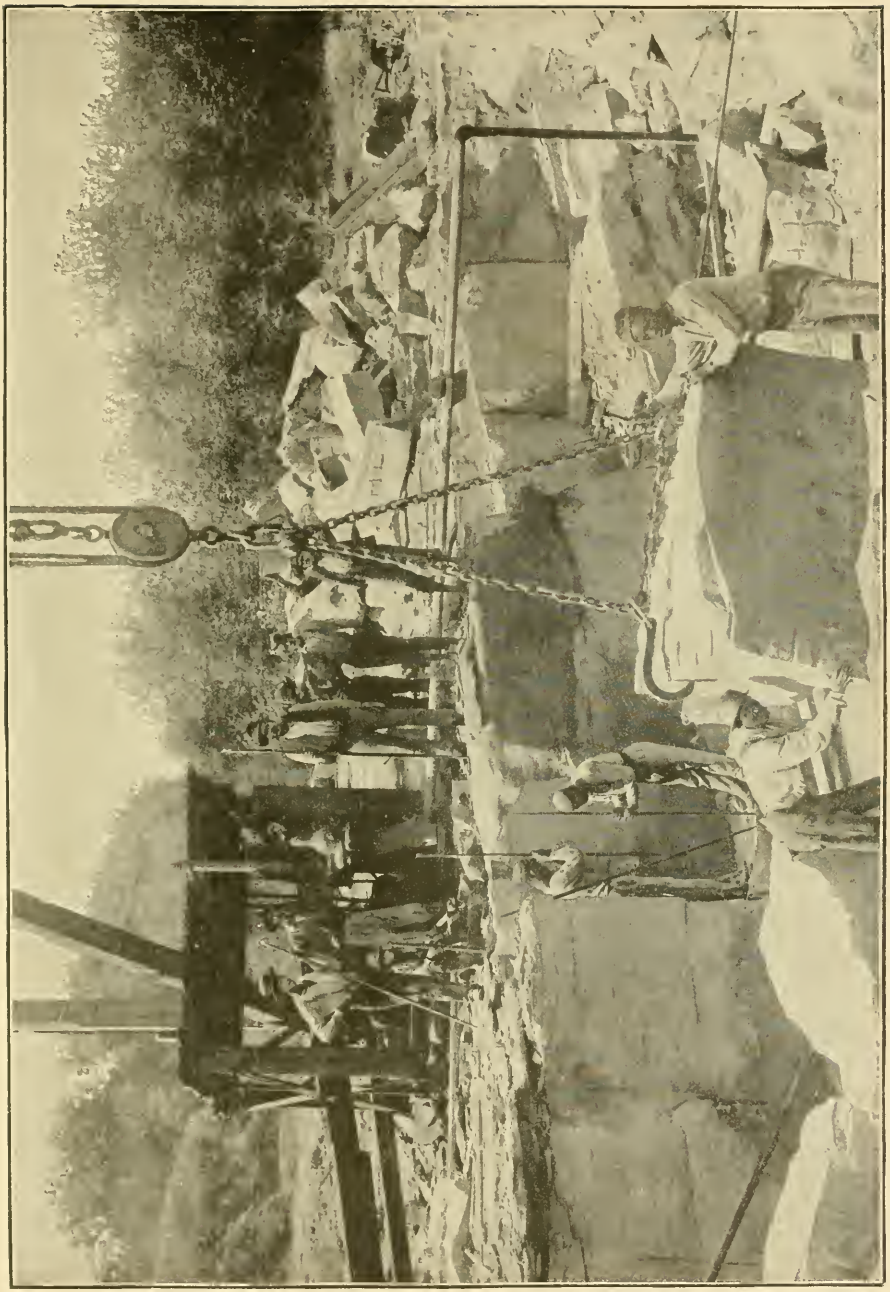
of this kind the plug and feather process applies very well, and the new system, when used, must be in the hands of an expert, or the loss will be serious.

Referring again to our stone step, let us imagine a case where this stair-way runs between two walls. We have here each step fixed at each end and free only on the top, the bottom, and one face. Let us assume that there is a back seam, that is, that the step is not fixed at the back. In a quarry, this seam, unless a natural one, should be made by a channeling machine. In order to throw this step out of place it must be cut off at both ends, and for this purpose the V-shaped holes are put in at right angles to the face. It is well, however, to put the first two holes next the back seam in a position where the grooves will converge at the back so as to form a sort of key which serves a useful purpose in removing the block after the blast. In quarries where there are no horizontal beds a channeling machine should be used to free the block on all sides and to a suitable depth, and then the ledge may be "lofted" by holes placed horizontally.

Where "pressure" exists in quarries, the new system has certain limitations. After determining the line of "pressure" it is only practicable to use the system directly on the line of thrust, or at right angles to it. It is much better, however, to release the "pressure" from the ledge by channeling, after which a single end may be detached by a Knox blast. It is well to bear in mind that the holes should invariably be of small diameter. In no case should the diameter of hole be over 1½ inches in any kind of rock. This being the case the blocks of stone are delivered to the market with but little loss in measurement. Every one knows that the buyer of a block of stone will figure its contents from the minimum measurement of its faces. A hole or groove at the top will shorten the measurement of the faces so that a good deal of stone may be quarried and even shipped to market without benefiting the quarryman, because it does not figure in the measurement. It is a noticeable fact that stone quarried by the new system shows very little evidence of drill marks, for the faces are frequently as true as though cut with a machine.

A matter of no little importance is the safety of the system. The blasting is light and is confined entirely within the hole. No spalls or fragments are thrown from the blast, hence the blasts may be made safely within city limits.





The popular idea that the system is antagonistic to the channeling process is a mistaken one. There are, of course, some quarries which formerly used channeling machines without this system, but which now do a large part of the work by blasting. Instances, however, are rare where the system has replaced the channeler. The two go side by side, and an intelligent use of the new system in most quarries requires a channeling machine. There are those who may tell of stone that has been destroyed by a blast on the new system, but investigation usually shows that either the work was done by an inexperienced operator, or an effort was made to do too much. Most good things are overdone. A quarryman who finds that a simple hole with a small amount of powder will release a large block in good marketable shape, is very apt to carry his enthusiasm so far that he attempts to use the system in places where it is not suited. Blasts are made where there are not sufficient free faces, and a desire to avoid the expense of a channeling machine leads to the destruction of a valuable stone.

A most interesting illustration of the value of this system side by side with the channeler, is shown in the Northern Ohio Sandstone Quarries. A great many channeling machines are in use there working around the new form of holes, and when used together in an intelligent and careful manner the stone is quarried more cheaply than by any other process that has yet been devised.

To a limited extent the system has been used in slate. The difficulty is that most of the slate quarries are in solid ledges where no free faces or beds exist; but it has been used with success in a slate quarry at Cherryville, Pa., since 1888. Among notable blasts made by this system are the following: At the Mica schist quarries, at Conshohocken, Pa., a hole 1½ inches in diameter was drilled in a block which was 27 feet long, 15 feet wide and 6 feet thick. The blast broke the stone across the "rift," only 8 ounces of Dupont black powder being used. At the Portland, Conn., quarries and the Middlesex Quarry Company, a single blast was fired by electricity, fifteen holes being drilled, with 2 pounds of coarse No. C powder in each hole, and a rock was removed 110 feet long, 20 feet wide and 11 feet thick, containing 24 200 cubic feet, or about 2 400 tons, the fracture being perfectly straight. This large mass of stone was moved out about 2 inches without injury to itself or the adjoining rock.

Another blast at Portland removed 3 300 tons a distance of 4 inches. Seventeen holes were drilled, using 2 pounds of powder in each hole, the size of the block being 150 x 20 x 11 feet. In a Lisbon, Ohio, quarry a block of sandstone, 200 feet long, 28 feet wide and 15 feet thick was moved about a half an inch by a blast. This block was also afterward cut up by this system into blocks 6 feet square. A sandstone bowlder 70 feet long, average width 50 feet, average thickness 13 feet, was imbedded in the ground to a depth of about 7 feet. A single hole 8 feet deep was charged with 20 ounces of powder, and the rock was split in a straight line from end to end and entirely to the bottom. A ledge of sandstone open on its face and two ends, 110 x 13 x 8 feet, was moved by a blast about 3 inches without wasting a particle of rock. Eight holes were drilled with a steam drill, three men being employed just one day, and 15 ounces of powder being used in each hole. A sandstone ledge, open on the face and end only, 200 x 28 x 15 feet, containing 84 000 cubic feet of stone, was moved half an inch by twenty-five holes each containing 1 pound of powder.

The reports from the Eleventh Census show that in the year 1889 about $53 000 000 worth of stone was quarried in the United States. Ten years prior to that time, as shown by the Tenth Census, the annual output was $18 000 000. This enormous increase has been made possible by improvements in the quarry which have cheapened the production of stone. Who would not use stone for buildings of all kinds, and who does not prefer it to wood, brick, iron, or any other material? Its great cost alone prevents its almost universal use.

DISCUSSION.

Leffert L. Buck, M. Am. Soc. C. E.—Mr. Saunders' interesting paper on quarrying is of a value disproportioned to its length. The subject is one that has rarely been brought before the Society, and as it is one in which our profession is as deeply interested as in any other, the paper deserves to be thoroughly discussed. Doubtless many of our members, who have not read the report of the tenth census on this subject, will be surprised at the rapid growth of the business in the country in the last few years.

The description of the process called "Lewising," reminded me of the first time I saw it used. While preparing the foundations of the Verrugas Viaduct, in 1872, there was a large boulder of excellent granite to

be broken up, and the foreman (a Scotch stone-cutter and mason) said he would break it by a "lewis-hole-blast." He caused two holes, 1½ inches in diameter and as close together as possible without running one into the other, to be drilled to a depth of about 7 feet. The holes were in the middle of the top and the plane of their axes coincided with that of rupture desired. The partition was broken out. Ordinary black powder was used and the break was almost an exact plane. It is not difficult to understand why the blast should act in this manner. There is greater surface for the pressure to act upon in one direction than in that at right angles to it, and of course the rupture will be parallel to the surface of greatest pressure, where the blast is about in the center of a stone of homogeneous structure. It is true that the effect of a three-cornered hole, such as an ordinary hand drill is apt to make, is to cause a blast to break the stone into three parts generally. This is also apt to take place where the hole is round, unless the greater part of the rock is on one side of the hole. I witnessed a fair proof of this in Pittsburgh in 1878. A "salamander" had been allowed to form in the bottom of a furnace stack and was broken by blasting. Roughly speaking, its shape resembled an army canteen and was 6 or 7 feet in diameter by about 2 feet thick in the center. A cylindrical hole was drilled in the center to a depth of about 17 inches with a ratchet drill. It was charged with dynamite, and broke into three nearly equal parts. There were slight incipient checks between the breaks which I was never able to account for satisfactorily. But that the piece should break in three places, is of easier solution. Of course we know it would break in the way that presented least resistance. As far as resistance to rupture alone is concerned, it would favor the piece breaking in two pieces, on or about some diameter, for the reason that each of the two fractures presents no greater resistance than each of the three, while the areas of the bursting pressures are respectively as 1 to about .866; moreover the direction of the bursting pressure is more favorable to the two fractures than to the three. But when the breaking is effected by a high explosive, the resistance of inertia comes in, and as this is but two-thirds as great in the three-fracture case as it is in the two, it becomes sufficient to outweigh all the other advantages which would favor the two fractures, and causes the three. Hence the higher the explosive energy the greater the number of fractures. In all cases the breaking is a tearing process from the hole outward. If this reasoning is true, it accounts for the advantage of the air cushion in quarry blasting. It might be questioned whether a still slower burning powder would not prove of additional advantage.

The census report referred to gives some statistics of cost of quarrying which are valuable.

M. W. Mansfield, M. Am. Soc. C. E.—In 1886, we advocated a trial of the Knox system of quarrying in the oolitic limestone fields of Indiana.

It did not meet with favor. I am of the opinion that this system will not work in oolitic stone for the reason that this stone is without seams and homogeneous and is highly elastic. All oolitic stone is now quarried with channelers and the block of stone is raised from the bed by driving small wedges in holes made with steam drills. These holes are usually 6 to 8 inches apart, 6 inches deep, and the time consumed in drilling each hole is about eighteen seconds. Skilled labor is not required in this work. I do not think the Knox system can be used advantageously and with economy in connection with channelers, but consider it an improved system for blasting which can be used only in quarries where blasting is permissible.

R. C. McCalla, Jr., Assoc. M. Am. Soc. C. E.—The Knox system of blasting has been in use since September 1st, 1891, on the Black Warrior River improvement at Tuscaloosa, Ala., in a quarry which probably could not be operated at a reasonable cost without the aid of this method. The quarry, a bird's-eye view of which is shown in Plate XCIV, is located on falls, in the bed of the Black Warrior River, adjacent to the sites of four locks which are now being built by the United States Government. This is the only quarry located in the bed of a stream of which I have ever heard, and was opened partly as an experiment and partly from necessity. The old quarry, located three-quarters of a mile above the present one, on the bank of the river above high water, had given out. The stone had become seamy and the stripping very heavy. There was no other available quarry site near the work and above high water, so that stone had either to be brought by rail from a distance or gotten from the river bed.

Lock 3, not yet begun, is located just above the present quarry, and considerable channel work is necessary for the lower approach to this lock, and as a coffer-dam had to be built and considerable work done at this point, in any case, it was decided to extend the dam further down stream, and by thus laying the bed of the river dry, make it available as a quarry.

The coffer-dam is 5 feet high, and, starting from the west bank just above the site of Lock 3, runs 200 feet across the current of the stream, which is 700 feet wide at this point; and thence 1 000 feet down and parallel to the current, to the head of an island which acts as an extension of the dam. The fall from the head of the dam to the pool below is about 7 feet at low water, and the area of river bed thus exposed is about 7½ acres.

The stone is a very fine quality of light blue sandstone, lying in ledges from 2 to 8 feet thick, which dip down stream at an angle of 7 or 8 degrees. In quarrying, a ledge is taken at its outcrop and followed down stream until it gets too deep to be worked to advantage. Then the quarry is stepped up to the next ledge above, which is followed in like manner.

There is considerable seepage water, which is controlled by a 3-inch

pulsometer pump located out of the way in one corner of the excavation and moved forward from time to time as the ledge gets deeper. Still, it is impossible to get rid of all the water immediately at the breast of the quarry, as the depth is constantly changing and varies at different points along the line of breast. As a matter of fact the stone is constantly quarried in from 1 to 4 feet of water, and this accounts for the great importance of the Knox system of blasting in this particular quarry. There is no chance to spring a hole, and whatever is done must be done by a single charging of the hole, the water entering immediately after the blast.

In a dry quarry it is practicable to detach a large block of stone, generally in some irregular shape, by repeated blasts in the same hole, beginning with a light charge and gradually increasing the charge until the block is separated from the mass, when enough powder may be placed in the crevice to throw the block entirely out of place without injury. It can then be cut up with ball drills, plugs and feathers. Were enough powder put in the old style of hole to detach the block at the first blast it would be almost certain to shatter the stone; and for this reason I do not think this quarry could be worked except by the Knox system without very great waste. Plate XCV gives an excellent view of a portion of the quarry breast.

At present, owing to the lack of machinery, all drilling in the Warrior River quarry is done by hand; but next season it is intended to introduce steam drills, which will reduce the cost of stone considerably. All drills used have the cross-bit, with two cutting edges at right angles to each other. A hole drilled by hand with a flat bit will invariably be somewhat triangular in cross-section, and a blast in such a hole will usually break the stone in three directions radiating from the corners of the triangle, resulting in more or less waste of stone; but there is not the slightest difficulty in drilling a round hole by hand if the cross-bit is used.

During the month of October, 1891, 200 cubic yards of backing and 600 cubic yards of dimension stone were taken from this quarry at a total cost of $1 597.73. Estimating the cost of backing at $1 per cubic yard, the dimension stone cost $2.33 per cubic yard. The quarrymen are negroes and are paid $1 for eight hours' work.

William L. Saunders, M. Am. Soc. C. E.—Mr. Mansfield, of Indiana, is quite correct in his statement that the Knox system of quarrying will not work to advantage in oolitic stone. He is, I think, mistaken in giving as one of the reasons for this that oolitic stone is without seams, homogeneous and highly elastic. It is not an uncommon thing to use the system in quarries where there are no seams. The channeler is used to make an artificial seam, which, in connection with the Knox hole, enables the quarryman to remove large masses of stone in marketable shape. The same objection to the Knox system of blasting in oolitic stone applies to any system of blasting whatever in soft stone. It is a well known

fact that powder is not generally used with success in removing banks of clay, because the material does not resist the force of the explosive, but it simply gives way around the hole, throwing out a mass of material and leaving a pocket in the shape of an inverted cone. A blast in a drill hole acts like a blow from a sledge hammer, and the higher the grade of the explosive the greater is the blow. We may strike a piece of pasteboard with a hammer and produce only a local effect, while the same force of blow, or even less, when applied to a piece of glass will shatter it to pieces.

Oolitic stone is quite soft and the easiest stone to channel that a machine has to encounter. It is not an uncommon thing for a channeling machine to cut 200 or 300 square feet of channel in oolitic per day, while the capacity of the same machine in marble is less than half as much.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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511.
(Vol. XXV.—November, 1891.)

THE BRAZOS RIVER HARBOR IMPROVEMENT.

By GEORGE Y. WISNER, M. Am. Soc. C. E.

WITH DISCUSSION.

The Brazos River and the harbor at its mouth have played a very important part in the development of the State of Texas, since the earliest settlements on the coast by American colonies; and the success which has recently been attained there in making it the deepest harbor on the coast, is likely to give it much greater prominence in the future. In 1824 General Austin made the harbor a port of entry for the colony. The commerce soon became of considerable importance and continued to increase until 1858, at which time a large amount of the river traffic was diverted to Houston over the railroad built west from that city.

During the days of the Republic, about 1845, a number of prominent citizens of Texas formed a company for the purpose of making a harbor and building a city at the mouth of the river. Nothing, however, was ever accomplished.

In 1857 the Legislature of the State, realizing the necessity of a first class commercial port on the coast, appropriated $60 000 to deepen the channel and improve the river from the mouth to the town of Wash-

ington, a distance of 250 miles. The improvement accomplished was only temporary, and a few years later the erection of railroad bridges across the river at several places, completely blocked navigation above tidewater. About the same date that the above improvement was undertaken, a company was formed and a charter obtained from the State Legislature for the purpose of deepening the entrance of the harbor and making it the eastern terminus of the Pacific route then being surveyed. Owing to financial trouble no work was ever done under this charter.

By an act of the Legislature of Texas, passed October, 1866, the Brazos Internal Improvement and Navigation Company was incorporated in the interest of the Houston and Texas Central Railway, for the purpose of improving the Brazos River for navigation by deepening the channel across the bar at the mouth, building a breakwater and dredging wherever necessary. The capital stock of the company was $1 000 000 and the charter was to continue for fifty years. The company had the power under its charter to condemn lands lying contiguous to their work, and to charge tonnage at a rate not to exceed twenty cents per ton of freight. Nothing further was done in the matter until March, 1872, when the owners of the Houston and Great Northern Railroad, then being constructed between Houston and Palestine, decided to join the Houston and Texas Central Railway people in the work of making a deep water harbor at the mouth of the Brazos River, and accordingly certain of the old directors resigned and a joint directory was formed from the stockholders of the two companies. The Houston and Great Northern Railroad obtained from the State Legislature a subsidy of $10 000 per mile, but this the following Legislature, in 1874, refused to recognize, on the ground that it had been procured by bribery. The failure to obtain this subsidy together with the financial panic of 1873 so crippled the company that no improvement work was attempted at the harbor. At the request of this company the Secretary of War granted a leave of absence for Captain C. W. Howell, of the U. S. Engineer Corps, to examine the bar at the mouth of the river, and submit plans and an estimate of cost for making a ship channel 20 feet deep from the river to deep water in the gulf.

Captain Howell was very sanguine of success, and in his report says:

"For a short time after the improvement of the bar, the harbor facilities afforded by the river will be sufficient, but it is, however, certain that the success anticipated for the work of improvement, giving the deepest entrance to the Brazos of any harbor on the Texas coast, must

attract a commerce that will rapidly outgrow the limited harbor room. You should, therefore, look forward to a system of docks communicating with the river by suitable locks, so as to have the whole river channel free for the passage of vessels up and down."

The plan submitted consisted of two jetties of closely-driven piles, starting from the headlands, at the mouth of the river, and converging so as to give an opening of 400 feet between the outer ends of the jetties on the crest of the bar, and estimated to cost $286 484. It was, however, fortunate for the promoters of the enterprise that the work was never undertaken, for, under the plans submitted, it would have been impossible to have carried it to completion. The teredo is so destructive on the gulf coast that piles driven in the open sea will not stand more than one season if unprotected, and the volume of discharge is such that the mean velocity of the current between jetties 400 feet apart would have exceeded 7 feet per second at times of floods in the river. Such a structure as proposed, subject to the action of the sea and river currents, would probably have been swept from its unstable foundation in a single night.

In 1874 Congress authorized a survey of the river from Waco to its mouth and including the bar. This survey was made during the years 1874 and 1875 by Captain R. B. Talfor, under the direction of Captain C. W. Howell of the corps of engineers. Captain Howell in his report on this survey repeats his recommendations made to the Houston and Great Northern Railroad Company in 1872, and submits the same plan for improvement then given. No further action was taken in the matter by the Government until 1880, when Congress made an appropriation of $40 000 to commence the work of construction in accordance with the plans of the U. S. Engineer Corps. The project was then referred to a board of engineer officers for revision, who discarded the old plans and recommended building parallel jetties of brush and stone, 700 feet apart, and estimated to cost, when completed, $522 890.44. The conclusion reached in the report was that possibly a channel 10 feet deep could be made and maintained across the bar.

In December, 1880, a contract was awarded for carrying the board's plan into execution, under which brush was to be furnished in place in the jetties for $6.75, stone ballast for $8, and small concrete blocks for $7 per cubic yard. Most of the work done under this contract consisted in revetting the east bank of the river, inside of the shore line, with brush mattresses ballasted with gulf sand. In 1881 Congress appro-

priated $40 000 more to continue the work, and another contract was awarded, under which brush mattress work was to be placed in jetties at $4.35, and gravel and concrete ballast at $7 per cubic yard. Under this contract the work on the east jetty was continued, but from all accounts very little of it remained long in place. In 1882, 1884 and 1886 $50 000, $10 000, and $18 750, respectively, were appropriated for continuing the work, all of which was expended in extending the east jetty, except a small amount of work in 1883 on the west jetty, which consisted of brush mattresses ballasted with plastic concrete made of natural cement in bags. The deterioration of the jetties was so rapid that in 1887 the engineer officer in charge reported that "the works heretofore constructed have disappeared, and have had no effect upon the bar." In other words twelve years of labor and an expenditure of $158 750 had produced no permanent result.

A survey and examination of the harbor entrance was made in 1887 by one of the most competent civil engineers on the gulf coast, who reported to the officer in charge that with a proper construction of jetties, a channel 16 feet deep could easily be made and maintained across the bar. Disregarding this the engineer officer reported after a long discussion that,

"With jetties 700 feet apart, the depth scoured by the greatest freshets would be something less than that found within the mouth of the river. It could hardly exceed 10 feet. How long it would take this depth to decrease to less than 6 feet, and whether the frequency of freshets is such that the necessary time would never be found, cannot be stated. All that can be said is that the attempt to maintain 6 feet is a very doubtful experiment. A study of the subject in the light of additional information shows the degree of improvement needed is impracticable, and that the attempt to make any improvement at all is a costly and doubtful experiment."

The officer seems to have relied entirely upon data in previous official reports which were very meagre and misleading. These state among other things that the river is a non-sediment bearing stream, because there is no delta at its mouth ; and that at low river it is like a series of pools separated by sand bars, which are often dry. Nothing could be further than this from the real facts of the case. The lower 50 miles of the river is a tidal stream, which for a low stage of water has a channel 15 to 50 feet deep for over 30 miles from its mouth. The total length of the main river is about 1 000 miles, and with its tributaries it drains an area of 36 000 square miles. The annual rain-fall along the coast is from 50 to 60 inches, in the interior 30 to 40 inches,

and for the whole drainage basin averages not far from 42 inches per annum. It is not probable that more than one-seventh of the rain-fall on the drainage basin finds its way into the gulf through the river, making the annual discharge about 501 811 200 000 cubic feet, or an average of 15 900 cubic feet per second. If this flow was a constant one, the problem of improvement would be a very simple one; but such is far from being the case. The period of low water in the river is of from one to three months' duration each year, during which time the discharge is only 1 000 to 3 000 cubic feet per second, while at the flood stage the discharge often exceeds 60 000 cubic feet per second. The amount of sediment carried in suspension by the river, varies from none at low water, to 4 ounces per cubic foot of discharge during freshets. The sediment is mostly fine alluvion, with a very small amount of sand.

The banks of the river are comparatively stable except on the lower 26 miles of its course, and nearly all the sand carried to the gulf by the freshets comes from the caving of banks on this lower reach. These banks are of clay for 10 to 20 feet in depth, resting on strata of fine sand. During high river these strata of sand become saturated, and when the pressure is relieved at low water the sand sloughs out and with the next rise is swept on towards the gulf. The banks, however, are of such a character that this trouble could be easily remedied, which would limit the character of sediment to that of such fine material that it would be swept far out to sea before finding a permanent resting place on the gulf bottom. The fineness of the sediment deposited on the bar and the strong littoral currents in the gulf, are the real reason that there has never been any permanent advance of the bar seaward, nor any delta formation at the mouth of the river.

The report of the Chief of Engineers for 1875 states that, "The floods and the 'northers' completely sweep away that portion of the bar formed during low water in the river, and drive the material so swept off into the deep water of the gulf." This is really the reverse of what does occur. The bar in its natural condition is formed by sediment deposited by the river during floods and by sand drift moving westward along the coast by the current and wave action. The past history of the harbor shows that the depths on the bar have varied but little except in cases when violent storms have closed the main channel, leaving depths of only 3 to 4 feet, which in some instances have lasted several

weeks before the river could break through the barrier in some new place. The extension of the bar seaward during freshets was not due to a pushing out of old bar material, but simply to an accumulation on the outer slope from the excessive load of sediment which the slackened currents were unable to carry further.

The low water season of fall and winter is also that of heavy easterly winds, during which the littoral currents westward are from one to three miles per hour. Under the action of the waves and of these currents the outer face of the bar has been annually worn away and the material distributed over the gulf bottom to the westward. This result is very plainly shown on the charts of the coast by the bending seaward of the 10 and 20 fathom curves immediately to the westward of the entrance of the harbor. In 1858, the 18-foot contour outside of the bar was 1 200 feet further seaward than in 1881, whereas after the big rise in March, 1889, when the present work was commenced, the position of the bar was approximately the same as in 1858.

Borings on the bar show that to a depth of 21 feet below mean gulf level it is composed of fine sand and shell. Below this sand and shell there is a stratum of clay about 2 feet thick overlying another of sand of 8 feet thickness, on a heavy clay bed, having about the same slope seaward as the gulf bottom. The bottom beyond the bar has approximately a slope of one in five hundred, and is mostly of clay and soft mud deposit from the river. During storms this soft material is disturbed to such an extent that the gulf becomes the color of the river at flood stage and causes changes of depth such that consecutive surveys often show differences of 1 to 2 feet over large areas.

The movement of the sand drift to the westward does not extend much outside of the 18-foot contour, and previous to the construction of the jetties had a tendency to narrow the entrance of the harbor during the low water season, by building up a sand point at the east side. The sand thus accumulated was swept out with each rise in the river, thus causing the cross-section of the entrance to be somewhat variable. Observations made when the entrance was thus choked by this drift action, led a recent board of engineers to the conclusion that the cross-section of the harbor entrance was so small that it was not worthy of consideration. Since the construction of jetties into the gulf this sand drift accumulates in a high concave sand beach to the east of the works, while

the material scoured from the river forms a convex bar to the south of the entrance.

The force of the gulf currents depends greatly on the direction and velocity of the wind. With strong easterly winds the currents are from 1 to 3 miles per hour westerly, while winds from the south and west produce very light surface currents in the opposite direction. The former, however, are the only ones of sufficient force to produce any material change in the conformation of the bar and of the gulf bottom beyond. The magnitude of these forces becomes very evident when we consider that eight to twelve million tons of sediment are annually deposited in the gulf from the river, and that no permanent change of the deep water contours occurs in front of the harbor.

The banks of the river for a distance of 50 miles up stream have approximately the same slope as that of the river at flood stage. The width of the river at 25 miles from the gulf is about one-half that at the mouth, and as the depth at flood stage is twice that at the entrance, the cross-section for the maximum discharge is approximately the same. This fact establishes the section of channel that can be made and maintained between properly constructed jetties at the mouth.

The slope of the river surface is not affected by the distance that the jetties are extended seaward, the only effect being an additional rise in the river for any given stage, equal in amount to that due to the slope in the jetty channel. With tidal harbors, however, the slope of the surface in the jetty channel is inversely as the length of the pass, and consequently on the gulf where the tides are very small (about 1 foot) and the distance to deep water very great, the plan of improving harbor entrances by confining tidal currents between jetties is a somewhat doubtful experiment.

The project now being carried out to make a deep water harbor at the mouth of the Brazos without Government aid was originated by Mr. W. M. D. Lee of Kansas, to whose enterprise and energy the success thus far attained is to a large extent due. Mr. Lee first visited the harbor in 1887, soon after the adverse report was made by the Government engineer, and was so thoroughly satisfied of the importance and feasibility of making a first-class harbor there, that during the following winter he secured the control of a large amount of land contiguous to the river near its mouth; and in March, 1888, organized the Brazos River Channel and Dock Company, and obtained a charter from the

Texas Legislature for the purpose of constructing, owning and operating a deep water channel from the waters of the Gulf of Mexico to the mainland at the mouth of the Brazos River. Mr. E. L. Corthell, M. Am. Soc. C. E., was employed to report on the feasibility of the project and to submit plans and estimates for doing the work. At Mr. Corthell's request, the writer, in March, 1888, made an examination of the harbor and vicinity, from which, with data obtained from the office of the Government engineer, his report was compiled and plans and estimates made. The only data of value obtained from the U. S. Engineers were a few comparative charts of the harbor entrance, and the elevations of the bench marks for a line of levels extending up the river 16 miles from the mouth. A very fortunate time, however, was chosen for making the examination, and the data then obtained have been subject to but slight corrections by the large number of careful observations since made.

The river at the time of examination was at about one-half flood stage; the plane of high water surface being well defined by marks on the timber along the banks. From a careful determination of mean velocity, discharge and slope, the co-efficient of roughness for "Kutter's formula" was determined for the section of the river used, from which, with the slope for high water, the maximum discharge was computed. Subsequent observations have since shown this determination to be in error less than one-half of 1 per cent.

The plan recommended for the improvement of the harbor was the construction of parallel jetties 600 feet apart, and extending into the gulf to a depth of 18 feet outside of the bar. The works were to be of brush mattress work ballasted with stone, and were expected to make and maintain a channel with a central depth of at least 20 feet. Mr. Corthell proposed in this report to construct the jetties from a trestle to be built seaward as the work progressed. The mats were to be built on tilting ways placed on a platform car, to run on a double track railroad laid on two parallel pile bridges, the piles being driven by an overhanging driver.* This plan, however, was not practical, as the mattress car would occupy the entire width of the trestle and consequently could not pass the driver to dump the mats at the end of the trestle as required. The delays which would arise from trying to carry on pile

* See *Engineering News*, June 22d, 1889.

driving and jetty construction in so limited a space would have made the progress very slow and the work very costly.

During the summer of 1888, the plans and method of construction were therefore changed by the writer. The system adopted consists in suspending the mat during construction, directly beneath the caps and stringers of the trestle, and when completed allowing it to slide down the piles of the trestle to its place on the gulf bottom, by simply loosening the suspending ropes attached to the caps and stringers. This method had the advantage over those previously used, that all work was built in place. Material could be transported over the trestle at all times, and the work carried on over any extent of jetty desired. On all work inside of bars, where not subject to heavy wave action, clay, shell, or gravel may be used for ballast and placed directly in the mats before binding, thus effecting a great saving of stone ballast, and at the same time making the jetties more impervious and safe from the destructive action of the teredo.

In August, 1888, a bill was passed by Congress authorizing the company to construct, own and operate such permanent and sufficient jetties and such auxiliary works as may be necessary to create and permanently maintain a navigable channel between the mouth of the Brazos River and the Gulf of Mexico. In December following, a contract was let to construct the jetties and create a 20-foot channel between the gulf and mouth of the river—the contractor to receive payments only as certain results were obtained. The work was to be done in accordance with the plans and specifications of the Chief Engineer of the company, but the only stipulation made as to the material to be used and the character of work, was that the jetties and auxiliary works were generally to be similar to those at the mouth of the South Pass of the Mississippi River. To the weakness and anomalous features of this contract may be ascribed the trouble that has harassed the company for two years and which on two occasions has nearly wrecked the enterprise. The fact that the contractor was to be responsible for results, should have carried with it the right to originate, design and construct the works, so long as the material used and work done were of a permanent character. On an enterprise of this kind, where unfinished work was liable to damage at all times, and the results expected were dependent on unknown quantities; with the design and direction of the work resting with one party, and the responsibility for results with another, a conflict was inevitable.

Mr. E. L. Corthell was appointed Chief Engineer for the Company, and in March, 1889, the writer took charge of the work as Resident Engineer. The change of plans made by the resident engineer by which the mats were constructed in place, and clay and shell used for ballast for the half mile of jetties inside of the bar, made a saving to the contractor of upwards of $25 000. The mats were constructed from a trestle of four rows of piles (spaced laterally to correspond with the width of the jetty) in bents of 16 feet each, with stringers of 10 x 10 inch timbers resting on caps of 8 x 10 inch timbers drift-bolted to the top of the piles. The timbers used for binding the mats were of 2½ x 6 inches, made continuous by splicing, and during construction were supported above wave effect by ropes made fast to the caps and stringers of the trestle. Sufficient brush was piled crosswise of these strips for half the thickness of the proposed mat, and it was then loaded with enough clay and shell to sink the entire mat. The brush for the completion of the mat was then placed at right angles to the former course and the whole firmly compressed between the mattress strips, with binders of wire or iron rods. The latter are preferable, but in this work a large amount of wire rope was used for the reason that the contractor had the material on hand and it was worthless for any other purpose.

Mattress work made in this manner is flexible, and when not subject to strain from wave action may be made continuous for any length, by simply lowering the mat by degrees as completed, so that it forms an incline from its final position in the jetty, to the unfinished portion on the suspending timbers. Where the mats were liable to be disturbed during heavy storms they were made in lengths of 100 to 300 feet, and allowed to sink quickly by loosening all of the suspending lines simultaneously.* Outside of the bar, where the force of the waves was sufficient to wash the clay and shell ballast from the mats, only stone was used for ballast, and it was generally placed on top of each mat after being completed.

No trouble was experienced on the half mile of work lying inside of the crest of the bar, but beyond this the force of the waves was such that a much more substantial structure was needed, and the contractor was surprised at the ease with which mats ballasted with 600 pounds of sandstone to each cord of brush were removed from the jetties by the

* See *Engineering News*, May 18th, 1889, for plates descriptive.

waves and the material distributed for miles along the shore. The clause of the contract requiring the works to be similar to those at South Pass, was interpreted to mean that they should contain one-third of a cubic yard of stone for each cord of brush, and while admitting the necessity of more substantial work, the contractor claimed that he was not legally required to do differently, and that all losses arising from weakness of the structure must be made good by the company. Under this state of affairs but little was accomplished, and after completing the jetties to the water surface for a distance of 3 000 feet from shore, all work was discontinued. The trestle was, however, unfortunately completed to a distance of 5 000 feet from shore on the west jetty, and 4 000 feet on the east jetty, which latter having no mattress to protect the bottom from scour, the waves cut a trench along the line of open piling, of such a depth that the amount of mattress work finally required was nearly double that which would have been necessary, if the work of jetty construction had been carried on simultaneously with that of the trestle.

In the latter part of June a heavy rise occurred in the river, and although at that time only 800 feet of jetties were in place, a channel 20 feet deep was scoured through the completed portion ; but the end of the works terminated so far inside of the crest of the bar, that the only result was to push the bar several hundred feet further seaward than was done by the rise of the previous winter.

During the summer an attempt was made to float the bonds of the company on the English market, and Sir John Coode, President of the Institute of Civil Engineers, was requested to examine the project and report on the probable result of the enterprise. Mr. J. C. Coode made a personal examination of the harbor and the plans for its improvement in December, and from the data thus obtained Sir John Coode made an elaborate report, in which he says :

"I consider the system on which the Brazos jetties have been laid down by the engineers of the company to be perfectly sound. The points on which I differ from these gentlemen are : First, the probable extent to which it will be necessary to project the jetties into the gulf ; and second, the amount of contraction requisite to maintain the standard depth throughout the year. The navigable depth, which is in my opinion the maximum likely to be maintained throughout the year by means of the works as proposed, has already been stated as 20 feet at mean low water, and a width of channel at that depth of 100 feet."

Mr. Coode was of the opinion that the jetties should be built out to the depth of 21 feet in the gulf in order to secure a permanent naviga-

ble channel of 20 feet, and that the entrance should be contracted to a width of 350 feet. Subsequent events have shown that a channel of such dimensions would not carry the volume of water discharged at the flood stage of the river without endangering the stability of the jetties, and that although the 20-foot channel has not yet been secured it is quite evident that the jetties will not have to be carried out to any such distance as he suggests. The report as a whole was very flattering, but unfortunately before it was made public the stringency of the English money market became such that no use could be made of it for the purpose intended.

At the end of the year the records of the engineer's office showed that there had been expended on the jetties, wharves and wing-dams $218 752, in addition to which the plant and buildings belonging to the contractor were claimed to have cost $70 198, or a total of $288 950, not including the personal expenses of the contractor. Of this amount, however, at least $50 000 may safely be charged to the account of lost material and worthless plant. The contractor's books showed a total expenditure of $327 000, or a difference of $38 050 in the two expense accounts for the same work. A compromise was finally made in January, 1890, by which the entire works and plant were transferred to the company, and a few weeks later Mr. Corthell resigned his position of Chief Engineer, after which the engineering and the entire management of the harbor and jetty construction were placed in the charge of the writer. An entire reorganization had to be effected, new plant and material obtained, and as the season of heavy easterly storms was at hand, but little was done for a month and a half except to make the work already in place more secure by loading the exposed portion with large rock. The trestle that had been built beyond the ends of the completed mattress work was such a wreck as to be of no use for future construction, and it had caused the waves to excavate a trench 16 feet to 18 feet deep for its entire length, where the normal depths would have averaged less than 10 feet previous to the construction. The outer face of the bar had cut away during the winter until it was approximately the same as when the work was commenced the previous year, with a channel depth on the crest of 8 feet at mean gulf level. After the completion of repairs to the old work, every effort was made to push the construction on the east jetty as rapidly as possible, so as to stop further scour along the line of the old trestle, concentrate the river current on the bar, and

form a breakwater for the protection of work on the west jetty from easterly storms.

In the latter part of April one of the heaviest rises in the river ever known began and continued for a period of six weeks, and during twenty days of this time the discharge exceeded 60 000 cubic feet per second and carried in suspension 400 000 to 600 000 tons of sediment per day. In addition to this vast amount of material, over a million cubic yards of clay and sand were scoured from the bed of the channel at the mouth of the river and through the jetties, but owing to its heavy character this was not carried by the currents much beyond the bar. The result of this rise was a channel 25 to 33 feet deep from the inside harbor to the outer end of the completed mattress work, 20 feet for 1 500 feet beyond, and 13 feet deep across the bar to the gulf. The face of the bar was built seaward 500 feet by the deposit of material on the line of the east jetty, and 1 300 feet on the line of the west jetty, making the direction of its front about forty-five degrees with that of the jetty channel. Beyond the bar, shoaling of one to two feet extended seaward nearly a half mile. To the east of the jetties no shoaling whatever occurred, showing beyond question that the gulf currents during the entire time of the freshet were to the westward. This is an important point, not only in regard to the problem now being solved at the Brazos, but also to that of every harbor on the coast.

During the months of May and June the east jetty was completed to a distance of 5 000 feet from shore, and work was stopped on the outer edge of the bar in a depth of 14 feet, and 500 feet inside of the 18-foot contour. In July and August the east jetty was ballasted with rock of 1 000 pounds to 3 000 pounds weight, and the mattress work of the west jetty completed to a distance of 4 000 feet from shore, and with the bottom course of mats to 5 400 feet, opposite the outer end of the east jetty. The first of September, the failure of the Potter Lovel Company of Boston, through which the Brazos Company transacted its financial business, completely demoralized everything and the works passed into the hands of receivers. Funds could not be obtained to complete the mattress work and build the concrete coping walls which the engineer had hoped to have in place in time to protect the jetties during the season of winter gales, and the entire force was disbanded except a small repair crew which it was absolutely necessary to retain to strengthen weak places in the work as they became apparent after storms. Before

financial relief was obtained, it finally became necessary for the Chief Engineer and his associates to put up their private funds for pay of labor to save the works from wreck, and even then the loss from storms was heavy. The jetties, as far as completed, were maintained above the plane of average tide until into March, 1891, when a series of heavy gales washed a large amount of the stone from the crown of the east jetty, leaving depths of 1 to 2 feet of water over the outer 2 000 feet of the work.

No rise of the river of any importance occurred during the fall and winter, and all changes of the bar were due to low river currents and littoral currents in the gulf. Previous to the extension of the west jetty in July and August, the channel across the bar under the action of southerly winds, which are almost constant at that season, shoaled to 11 feet, but no changes worthy of note occurred on the outer slope of the bar. When mattress construction was carried out to the crest, this shoaling was stopped and the channel under the natural forces of the river again deepened to 13 feet. With the commencement of easterly winds in September, rapid erosion became apparent on the outer face of the bar, and continued throughout the winter to such an extent that the 20-foot curve was only 100 feet outside the jetty entrance, and the depths beyond were from 1 to 2 feet greater than at any time since the commencement of the work in 1889. The bar to the south and west of the west jetty was not entirely cut away, and probably never will be. The charts of the South Pass entrance of the Mississippi River show a similar bar extending 1 500 feet beyond the jetties, and at the Sulina mouth of the Danube similar results are apparent, though not to so great a degree.

In its original state the river was about 700 feet wide at its mouth and 14 feet deep, shoaling toward the gulf to 6 feet on the bar, and deepening up stream to a depth of 45 feet at the bend of the river, one-half mile from the mouth. The river channel above the jetties has been corrected to an average width of 475 feet by wing-dams, so located that the new channel forms a tangent to the curve in the river above, thus avoiding any liability of a cross-over bar being formed. This straight reach of river extending to the outer end of the jetties, a distance of 1½ miles, is such that the momentum of flood discharge of the river (after removal of the bar) will probably produce scour for at least 1 500 feet beyond the end of the works.

To prevent undermining by the formation of a deep channel near the jetty walls, spur dikes 40 feet long were constructed at intervals of 400 feet along each side of the jetty channel. The deposits produced by these spurs have built solid walls of mud along each jetty, making them impervious and safe from any danger of undermining or destruction by the teredo.

The inner edge of the bar in its original state was about 3 000 feet from shore on the channel line and united with the shore a half mile west of the entrance. Soon after the completion of the west jetty to the bar, the portion to the westward of the works was built up by deposits and wave action and became a permanent shore line. The pool formed by this new shore line and jetty soon silted up so as to be bare at low tide, thus making the inner 3 000 feet of the west jetty practically a river bank.

No river deposit has accumulated to the east of the works, but the movement of sand drift along the shore has formed a high concave beach in the angle of the east jetty, and extending 1 200 feet seaward. Sand is banked against the outside of the jetty for 3 000 feet from shore, but slopes down to the original depth at a short distance from the works.

The method of mattress construction for 1890 was similar to that of the previous year, except that half-inch iron rods were used for binders instead of wire. Compression was given to the mats after construction, by passing the ends of the rods through holes in the upper binding strip until a hold could be obtained with a shackle bar, with which pressure of about one ton was applied to the mat by lifting the rod through the strip, and afterward fastening it firmly in position with an iron wedge driven into a washer resting on top of the mattress strip. The piles for the trestle were driven with an ordinary overhanging driver of sufficient reach to drive bents 16 feet in advance of the completed structure. With average weather, three bents (48 lineal feet of trestle) were completed per day, and including lost time cost eighty cents per lineal foot for construction. From an average for the season, it was found that the labor required in the construction of mats was one and one-third hours for each cord of brush used, and one hour for each ton of rock distributed in place in the jetties. The brush and the rock were delivered alongside the works on barges. Two cubic yards of brush as

measured on the barges, when compressed in the mats and consolidated with heavy stone ballast, made one cubic yard of completed jetty work.

The following table gives the cost of one cubic yard of completed jetty as determined from an average for the season on the outer 2 000 feet of the works where exposed to heavy sea action:

Trestle	$0 20
Scaffold and mattress frame	0 05
Brush	0 70
Labor on mat	0 10
Rock ballast	1 40
Labor placing ballast	0 08
Engineering and superintendence	0 07
Cost of 1 cubic yard in place	$2 60

The brush cost $1.50 per cord delivered on board of barges, and the rock ballast $3.50 per ton on barges alongside of the works.

In April, 1891, a reorganization was effected by which the contractor, who on retiring from the work the previous year, stated that the enterprise was practically a failure, again took charge of the management and the writer resigned his position of Chief Engineer. The work is so far advanced and the results so satisfactory that no change of plans is contemplated. Mr. A. E. Kastl, M. Am. Soc. C. E., who was assistant on the work during the first year of construction, has been appointed engineer, and will carry out the original plans as far as the means placed at his disposal will allow.

No rise in the river of sufficient duration to produce deepening of the channel occurred until the latter part of April, previous to which, as before stated, the works had deteriorated to such an extent that the outer 2 000 feet of the east jetty was 2 feet or more below mean gulf level. A large amount of the river discharge was consequently wasted laterally over the top of the jetties, but even with this loss of force a 15-foot channel was made across the bar, and the inner face cut away so that the 18-foot contours on opposite sides of the bar were only 400 feet apart. This channel has since increased in depth to 16 feet from the effect of the currents across the bar at a medium stage of the river. The bar in front of the west jetty was pushed seaward several hundred feet, but to the east of the axis of the jetty channel no change of position took place. A deposit of 1 to 2 feet of soft mud was made on the outer slope and beyond, which is now rapidly disappearing. It is quite evident

that if the jetties had been well above the water surface for their full length, the velocity of the current beyond the entrance would have been such that no deposit would have been made for a considerable distance in front of the works.

Under the plans adopted at the commencement of the work in 1889, it was the intention, as soon as the jetties were sufficiently settled, to build concrete coping walls on the portions exposed to wave action, of a similar construction to that designed and built by the writer at South Pass the previous winter.* The financial trouble of the past year prevented this work being done, and resulted in losses to the work fully equal in amount to one-half that which the walls would have cost, and which delayed the desired result of obtaining a deep water channel for at least one year. It is a notable fact that at South Pass previous to the construction of the concrete coping walls, a depth of only 24 feet was obtained, which in less than six months afterwards increased to over 30 feet. At the Sulina mouth of the Danube only 15 feet were maintained previous to consolidating the jetties with concrete blocks. This depth was increased to 22 feet by the natural forces of the stream very soon after both jetties were built to the same distance seaward and capped with a concrete superstructure. A similar result may be expected at the Brazos so soon as the jetties are thoroughly consolidated and strengthened with a proper superstructure. Until then they will be in constant danger of being wrecked by storms, and cannot be maintained in effective condition to give a much better channel than that now existing there.

Some of the more important principles governing jetty and harbor construction which became very apparent at different stages of this work may be worthy of note, and are briefly summarized below :

First.—Jetties, to produce a maximum result at a minimum expense must be completed beyond the bar in a single season if possible. The bar then acts as a submerged weir with the strongest current on the outer crest, thus transporting all eroded material to a safe distance from the entrance.

Second.—Delays cause great increase in the cost of construction from damage to the works by storms, and the much greater distance the jetties have to be built seaward.

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* For plates and description see *Railroad Gazette*, September 13th, 1889.

Third.—The success of jetty improvements depends largely on the existence of strong littoral currents in front of the harbor entrance, otherwise the advance of the foreshore and bar would soon close any channel obtained.

Fourth.—In jetties at the mouths of rivers, the strongest currents are at the outer end of the channel, and in no case is it necessary to build the works to a greater depth beyond the bar than that required in the channel.

Fifth.—When scour takes place in the channel bed its action is first noticeable at the lower end of the section and gradually works up stream.

Sixth.—In fresh water streams flowing directly into the gulf, salt water currents up stream often exist where the surface current is flowing seaward, and consequently surface velocities are no sure measure of the discharge or scouring force. When the east jetty was completed 2 000 feet in advance of the west, the main current at ebb tide flowed past the end of the unfinished jetty at nearly right angles to the jetty channel. The shoaling which then took place on the bar, and the subsequent deepening when the jetty was extended, very plainly indicate that one jetty would not be very effective for channel making at ports of this class.

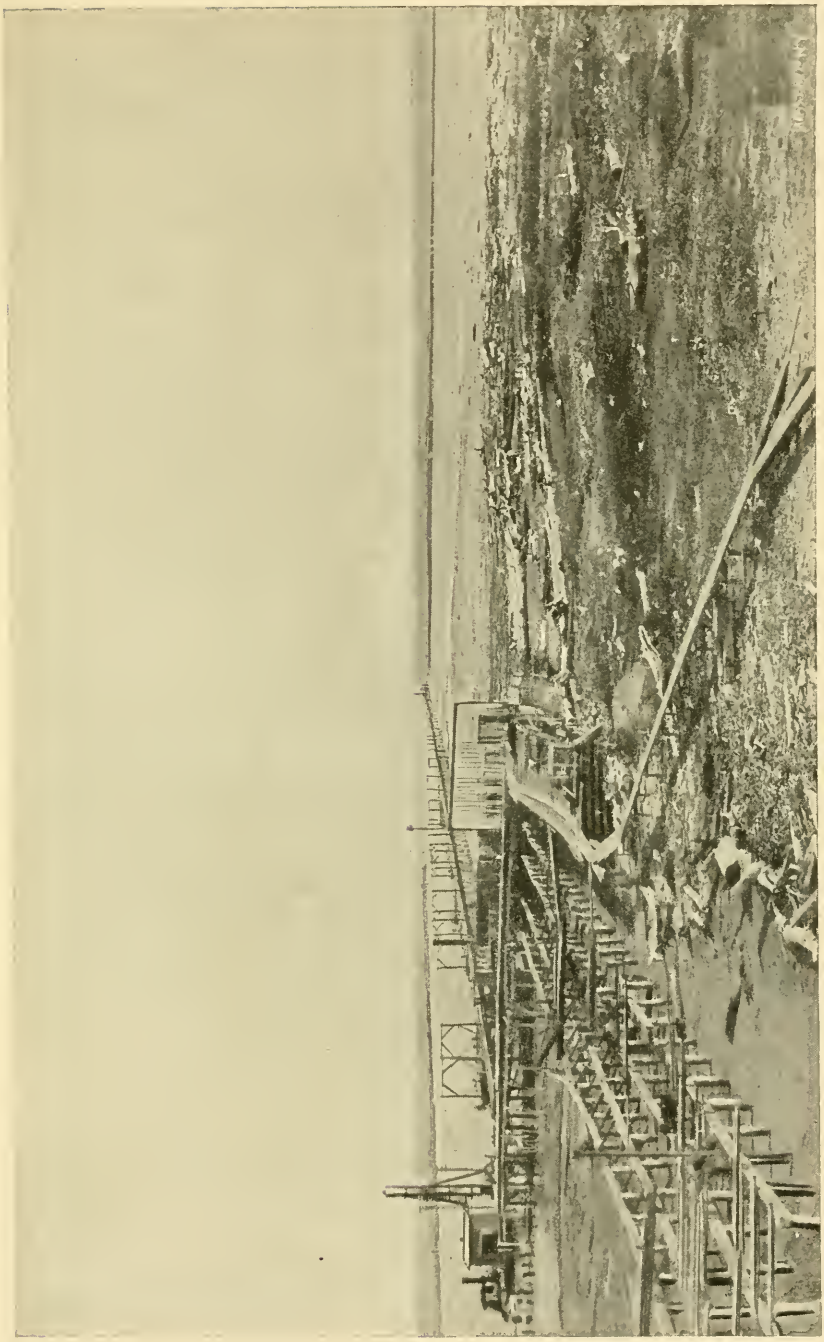
Seventh.—To insure successful results for an enterprise of this character the engineer must not only have control of the plans for improvement and design of the structure, but also of the methods, material and rapidity of construction.

Description of Plates.

Plate XCVI. View of the pile-driver, trestle and suspended mats near the outer end of the west jetty.

Plate XCVII. View of the jetty channel from the head of the west jetty, showing the new shore-line to the west of the works, and in the left foreground, the mud deposit produced by wing-dams.

Plate XCVIII. View of the river looking down stream, near the upper side of the new town site. The width of the river at this point is about 500 feet, with a central depth of 23 feet. The wharves for the new town are being built along the concave bank shown on the left.



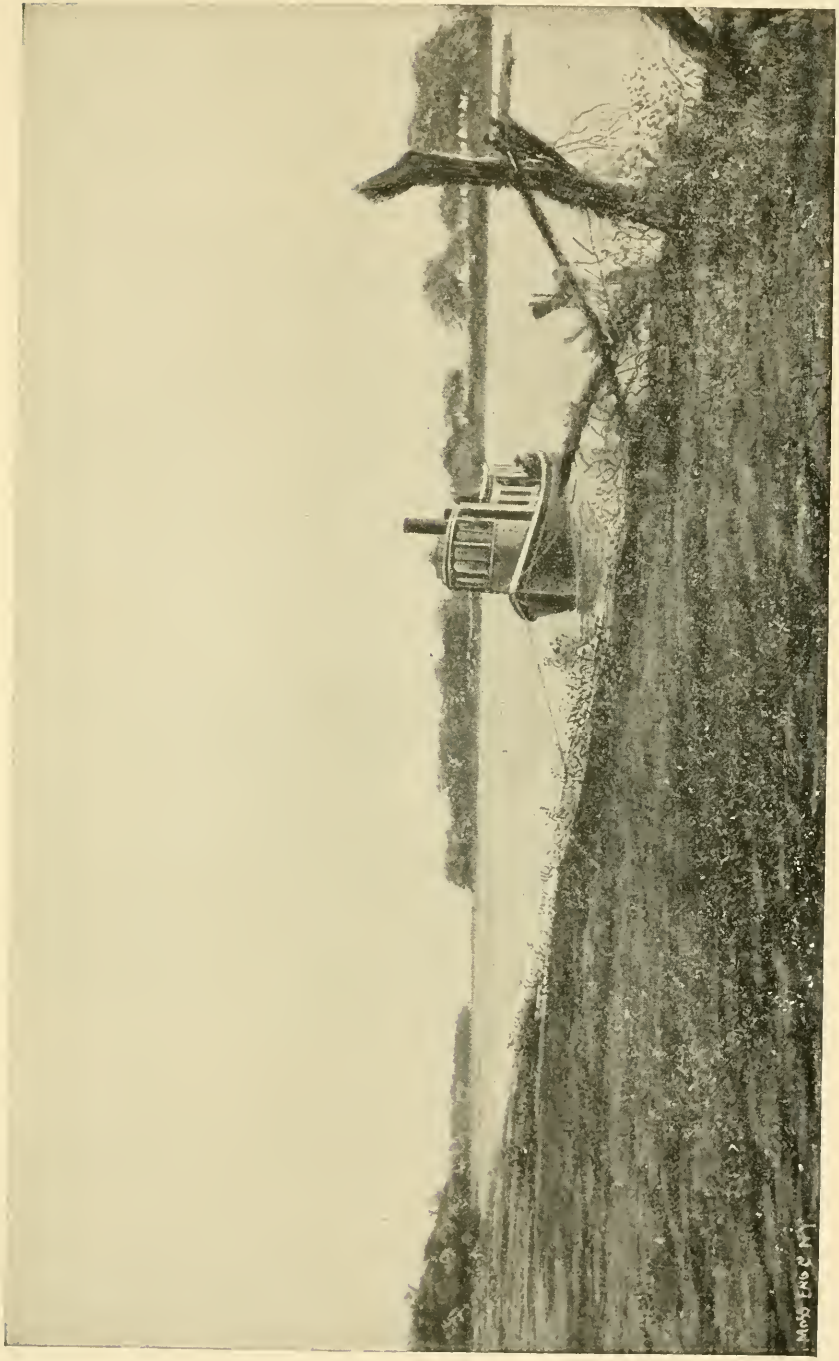


PLATE XCIX
TRANSACTIONS OF ENGRS
VOL XXXV No 51
WISNER ON BRAZOS RIVER
HARBOR IMPROVEMENT

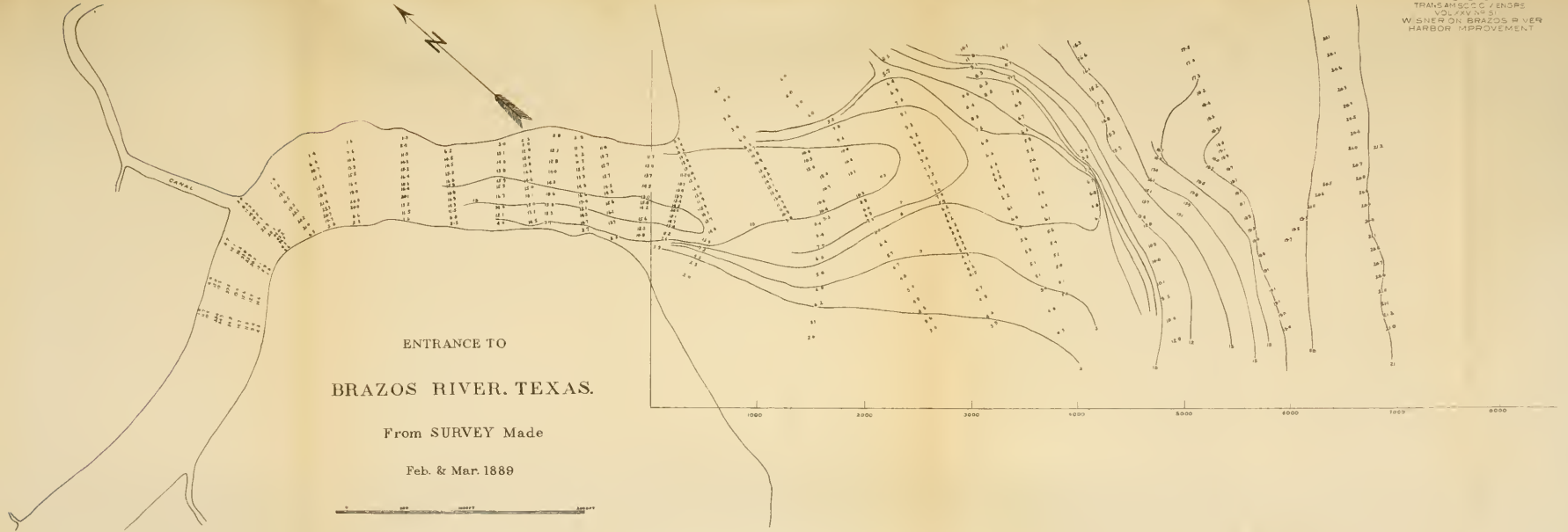
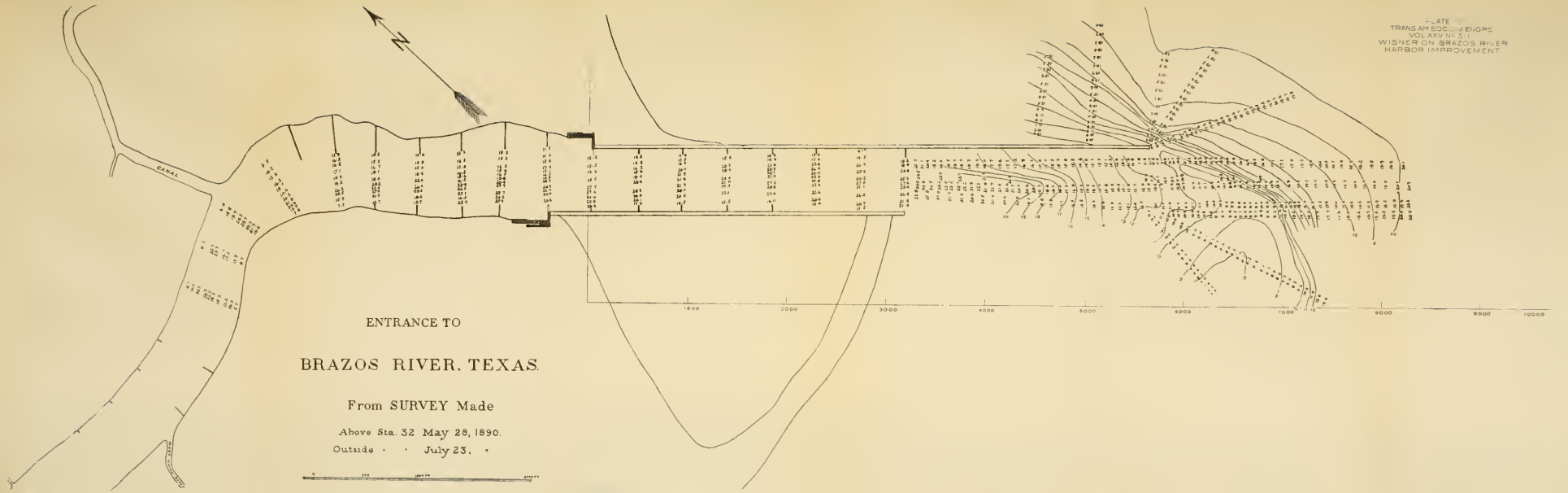


PLATE
TRANS AM SOC CIV ENGRS
VOL XXV NO 511
WISNER ON BRAZOS RIVER
HARBOR IMPROVEMENT



ENTRANCE TO

BRAZOS RIVER, TEXAS.

From SURVEY Made

Above Sta. 32 May 28, 1890.

Outside · · July 23, ·

PLATE CI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. Nº 511.
RIPLEY ON TIDAL PHENOMENA
IN GALVESTON HARBOR.

TIDAL PHENOMENA OF
GALVESTON HARBOR
sketch
showing location of tide gauges

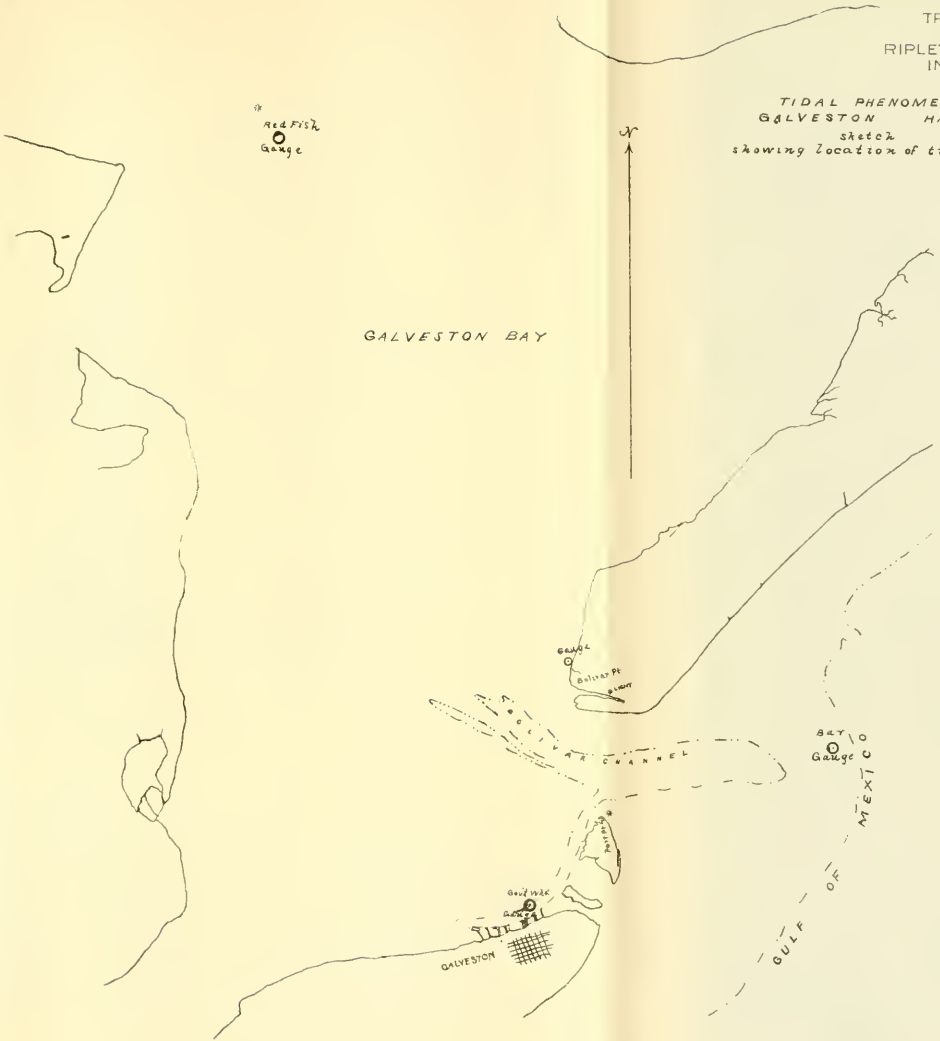


PLATE CII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV No 511
RIPLEY ON TIDAL PHENOMENA
IN GALVESTON HARBOR.

Tidal Phenomena, Galveston Harbor

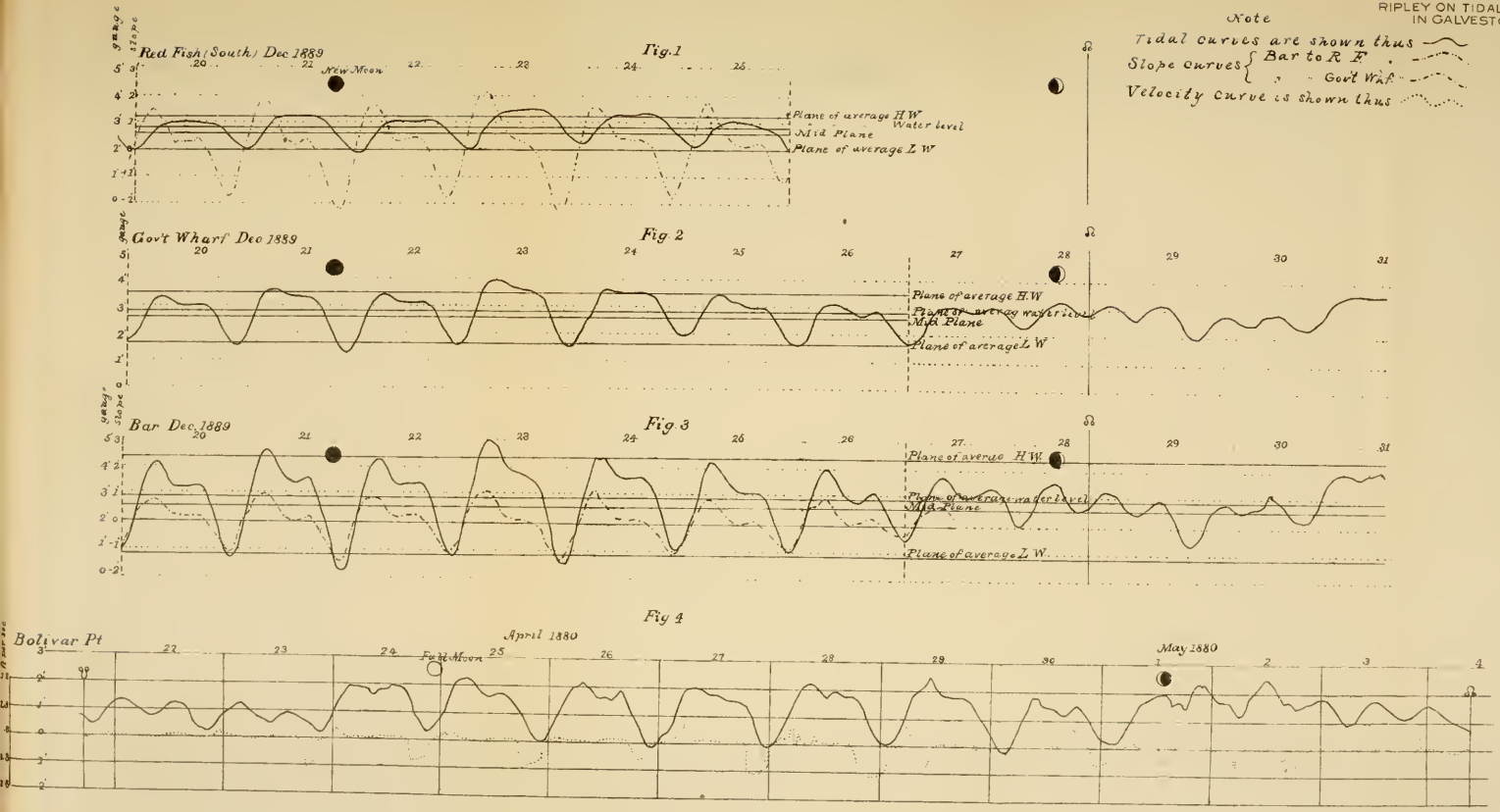


Plate XCIX. Chart showing the depth of channel in the mouth of the river and on the bar just previous to beginning the jetty construction.

Plate C. Chart showing the improved river channel above the jetties, the extension of the bar seaward caused by the freshet of May, 1890, and the position of the jetties at the time of the subsequent shoaling of the channel. The stations are 100 feet apart and number from the head of the west jetty. Six months after the date of this survey, erosion had taken place to such an extent that the 20-foot curve was only 100 feet in front of the jetty entrance.

DISCUSSION.

G. H. Mendell, M. Am. Soc. C. E.—It appears from official reports that depths upon the bar at Brazos River, in its natural state, have varied from 2.5 feet to 8.5 feet; that changes have been very sudden and of constant recurrence; that in 1887 the maximum draft of vessels trading there was 4½ feet; that during the previous September no vessel was able to cross for want of depth; and that the magnitude of the bar has varied greatly, as evidenced by the positions of the interior and exterior 12-foot contours—the interval having been found as follows: in 1858, 2 100 feet; in 1875, 4 650 feet; in 1881, 1 750 feet; in 1882, 1 200 feet, and in 1887, 4 500 feet. The alignment of the bar channel has also varied very much. After a freshet it is direct. Subsequently it undergoes frequent changes, being at times perpendicular to a former position.

There is material discrepancy between the description of the lower portion of the river, as given in official reports, and in the author's account. But both so far as they relate to depth are very general; one stating "about 12 feet with pools of 18 feet," the other "15 to 50 feet."

It appears from the paper that the following conditions exist, namely; almost an absence of tide which, however, must supply most of the low water discharge, reported to be as little as 1 000 feet per second; a duration of the low water season from one to three months; freshets of short duration carrying 60 000 cubic feet or more per second with one-two hundred and fiftieth of their weight in sediment, estimated for the year at 10 000 000 tons, a very extraordinary variation in flow, the maximum

being of short duration and sixty-fold the minimum and even more, a current to the westward 1 to 3 miles an hour due to winds, an unfavorable action on the bar during the prevalence of summer winds from an opposite direction; a low slope of the bed of the ocean, namely, 1 foot in 500, great destructiveness by the teredo, costliness of stone involving construction of jetties mainly in brush. The resultant of these conditions thus baldly stated cannot be considered as altogether promising. If they could be defined in their relative values, favorable and unfavorable, perhaps an *à priori* judgment of a definite character might properly be made as to the measure of success likely to attend this improvement. But this is not now possible.

The circumstances of an uncompleted work, while interesting and suggestive, perhaps, cannot carry the lessons which are to be derived from consideration of the conditions found to exist several years after completion. At this later period we shall be informed whether or not the recurrence of former conditions, so generally appertaining to bar improvements of non-tidal rivers, shall here obtain.

If the jetties prove to be permanent, escaping the destructive action of the teredo, the effects of the storms of the Gulf, and other hostile agencies that may exist, we shall always expect to find between their lines a sufficient depth, and if the exterior forces working for good shall, year by year, sufficiently remove from the front of the jetties the 10 000 000 tons of silt brought by the river and the sand drifted from the east, there may be 20 feet of water over the bar, as is expected by the engineer. But if the westward movement at the bar be unable to take care of the material presented to it, whatsoever be the depth obtained by a speedy construction of jetties, we may expect, after an interval of time proportioned to the resultant amount of deposit, that something like the old entrance will be restored a few thousand feet in front of the end of the jetties. This interval of time would, of course, be greater, other things being equal, if the slope of the ocean bed were greater. The final issue appears to be rather in the hands of nature than within control of man.

It is not important, but it may save misconception, to say that the method of construction of mats designed by the engineer of this work in 1888, viz., by suspension from the timbers of the trestle, stated by him to have been successfully used by United States engineers on the Pacific Coast for the past two years, was first applied here on the harbor improvement at Yaquina Bay in the spring of 1883. It has been continuously in use by the United States engineers for the past eight years at Yaquina, and at the mouth of the Columbia River. The method is mentioned in the Chief of Engineer's report, 1883, page 2066, also in 1884, page 2265.

O. H. Ernst, M. Am. Soc. C. E.—The questions involved at the mouth of the Brazos are simply these: how large a channel will a river confined

by jetties carve for itself, the discharge of which varies from over 60 000 to less than 800 cubic feet per second; and what portion of the maximum channel secured during the floods can be maintained during low water, in the face of wave-action upon a low and sandy sea-coast? The engineers who attempted to answer these questions by theoretical discussion obtained widely different results. According to some there would be no difficulty in making and maintaining a channel 20 feet deep and 100 wide, or even a greater channel. According to others a channel 10 feet deep and 100 feet wide could be obtained with jetties 700 feet apart, and a greater depth for that width with jetties closer together, but it would be impracticable with any degree of contraction to maintain a depth of 20 feet and width of 100 feet—the least dimensions at all suitable for a naval station or harbor of refuge—and it was even possible that under a certain combination of circumstances, not uncommon to the locality, the dimensions might be reduced to 6 x 100 feet, that is that there would be no improvement at all. Somewhere between these two extremes there might be some permanent improvement, but the degree of it was so uncertain that the whole enterprise must be pronounced a costly and doubtful experiment. The result of the practical demonstration by the actual construction of jetties which has been undertaken, has therefore been, and is still, awaited by them with much interest.

Further theoretical discussion at this time would seem to be profitless, but all recent and detailed information about the progress being made in improving the channel is interesting. It is to be regretted that the only map published with this paper is from a survey made more than a year ago, and that the width of the 16-foot channel obtained during the freshets of two or three months ago is not given. It is even difficult to ascertain with accuracy the clear width of waterway between the jetties. At one place in the text the distance between the jetties is mentioned as 600 feet, but subsequently reference is made to a change of plan. Scaled from the map the distance is about 560 feet, reduced by the spurs to about 450 feet. A depth of 16 feet, or a much greater "central depth, without regard to width," proves nothing for or against the correctness of either of the opposing opinions. The dimensions of channel which can be maintained throughout the year and every year, are what mark the degree of improvement of which the locality is capable. The season of low river and heavy cyclonic disturbances of the gulf is just approaching. It will be interesting to know what effect these circumstances will have had a few months hence, whether there will have been any decrease in the dimensions of the channel, and if not whether such changes will have occurred in the shore-line as will render such decrease more or less probable upon the recurrence of a similar season. It is to be hoped that the engineers familiar with the work will inform the Society at that time, and will furnish a map showing the hydrography to date.

The operation of constructing and placing the mattresses was ingenious and possessed some novel features. Inshore the use of that material appears to have been successful. Whether it will prove to be so at the sea ends of the jetties, remains to be seen. The stripping by heavy seas of the ballast from the top of the east jetty, which was in pieces weighing only from 1 000 to 3 000 pounds, was to be expected. Much heavier blocks are needed here. The paper contains many constructional details, for which the author merits the thanks of the Society. The general question, however, of the practicability of making a permanent channel 20 feet deep and 100 feet wide, is where it was before the work began. Nothing has occurred as yet to controvert the opinion expressed in the report which caused the Government to abandon the improvement.

It would, perhaps, have been better if the author had deferred the publication of his paper until something definite had been obtained from the practical test.

ARTHUR J. MASON, M. Am. Soc. C. E.—Mr. Wisner's paper on the Brazos River improvement, brings out strongly the value of littoral currents in aiding the improvement of channels at harbor entrances. The function of the channel current when contracted, is to loosen and hold in suspension the solid matter, the function of the littoral current to transport that material out of the way, in common parlance, to waste or spoil it. In a recent visit to the mouth of the Brazos, the part played by the littoral current was very strikingly shown by that lightest of all silt, drift wood, the angle formed by the west jetty and the west shore being covered for perhaps a hundred acres with a labyrinth of tree trunks brought down by the river, the beach eastward showing very few.

The success of the work at the mouth of the Brazos, and at other places, shows that the actions of the sea currents and other forces are no longer to be regarded by engineers as whims of nature, too complicated, too arbitrary, too intangible for human observation; that these forces are capable of subtle, careful analysis, based on close, long extended observation, that the treatment of harbor channels can be undertaken with the same confidence and success as the many equally great enterprises of which one hardly hears, but that they are proposed, under contract, and successfully completed.

LEWIS M. HAUPT, M. Am. Soc. C. E.—The question of securing a deep water port on the Texas coast is one of great interest to the entire country. While engineer officer of that district after the war, I was impressed with its importance, and subsequently I have given considerable attention to the various projects and efforts which have been made to this end. In this paper, Mr. Wisner has given a carefully prepared résumé of the history and operations at the mouth of the Brazos River, showing very fully the attempts and failures, and stating the reasons therefor, thus making it an instructive example for the use of maritime engineers. Much credit is due the syndicate for the courage exhibited in undertaking a project which had just been so entirely condemned as

impracticable, but it must have been evident to them, as to others, that a correct diagnosis had not been made of the physical conditions, and that the case was not therefore altogether hopeless.

Of the several inlets which indent this coast, the Brazos is the only one between Sabine and Corpus Christi which is not dependent entirely upon the tidal volume for its action. Having no interior lagoon or estuary, it belongs to the class of rivers which debouches directly into tidal water, and which, therefore, offers the most favorable conditions for improvement by parallel jetties properly spaced and located.

With reference to this first point, viz., width between jetties, the interval is admirably adapted to secure the best results for this particular river, but, as to the location, it is as yet incomplete and subject to change as to length, according to varying conditions. The original length has, I believe, been already exceeded in consequence of unforeseen delays in the prosecution of the work. The direction was selected with reference to the prevailing seas, requirements of navigation, discharge of fresh water and the littoral currents, and seems to be a happy compromise between these somewhat conflicting conditions. The case is peculiar in its physical conditions and in its resemblance to the few successful instances of jetties in pairs at the mouths of rivers with weak tides, but where there is a large tidal movement, even though the range be small, and especially where there is a prevailing direction of littoral drift.

I have advocated for some years the construction of a single line of defensive works to protect the channel on the outer bar of inlets from the traveling sands, as being a better expedient for tidal entrances than the usual pair of jetties intended to produce concentration of ebb currents, and I believe that the more recent practice has fully confirmed the correctness of these principles. One of the latest instances of this kind is to be found at the mouth of the Columbia, in Oregon, where the single jetty which has been built on the south side of the entrance has intercepted the drift of sand and shingle and thus improved the depth several feet, although the bar has been pushed seaward and is still in an unstable condition, because of the improper form and location of this work. The original project contemplated another short jetty from Cape Disappointment to increase the contraction. This should not be built, as it would still further increase the shoaling which has taken place inside of the capes and reduce the tidal prism entering the estuary.

During a recent visit to the Pacific coast harbors I have learned of another instance of the failure of the twin jetty plans as designed for Coos Bay, Oregon, where the entrance has been so obstructed that the vessels which formerly sought shelter there now avoid it entirely as being too dangerous.*

* This is on the authority of one of the largest shippers of the Pacific coast.

Of the attempt made to obtain deep water at Galveston by twin jetties located 7 000 feet apart, I need only say that under the present plan it must result in a calamitous failure, as such jetties cannot co-act to produce scour, and are too remote from each other to protect the channel.* Even those at Sabine Pass are admittedly too far apart to produce the desired result. While the other inlets at Corpus Christi, Cavallo and Aransas have been abandoned as incapable or unworthy of improvement; and yet it is possible with a comparatively small scour to create a good channel over the bar.

The results thus far attained at the Brazos River are evidence of the success which may be expected to attend all well designed and rapidly executed projects of this character, and the projectors are to be congratulated upon the success which has thus far attended their efforts in the face of many discouragements and difficulties.

Mr. H. C. Ripley.—The admirable paper of Mr. Wisner on the Brazos River improvement will certainly be read with the greatest interest by all engineers in this country who have given any study to this branch of engineering; for, without doubt, the success achieved at the Brazos is the most signal of its character since the accomplishment of the South Pass improvement. Our national legislators will also be interested in this paper if they reflect that this work has been accomplished by private enterprise under exceedingly embarrassing financial conditions within three years of the time of the passage of the bill in Congress authorizing the company to commence operations there.

Mr. Wisner's paper gives the impression that no guesswork was permitted to usurp the place of exact information regarding the essential features of the problem to be solved; and, having this information, the engineers were able to express unlimited confidence in the success of their work.

A board of engineers had previously expressed its opinion concerning a plan which it had devised for the improvement of an important harbor in these words: "It will be seen that the board does not attempt any prediction of the precise depth the jetties will maintain. Such predictions can best be made by those ignorant of experience in tidal entrances elsewhere, and having great confidence in the credulity of mankind."

It is unfortunate that the author did not furnish with his paper a chart of the last survey of the Brazos entrance. During a recent visit there the writer had an opportunity of inspecting this chart. A comparison of it with those of the Sulina mouth of the Danube and South Pass, Mississippi River, shows a similarity almost as great as that which charts of two different surveys of the same place would show. In each case we have a sediment-bearing stream of fresh water debouch-

*See Franklin Institute Journal, October, 1891, for a brief history of this work, with statement of results to date.

ing into a sea of salt water, a littoral current flowing past the mouth in one direction and a similar treatment by jetties extending normal or nearly so to the direction of the littoral current. And we find in each case the deep water close to the end of the jetty on the side from which the littoral current comes, an immense shoal extending far out beyond the end of the jetty and a new shore line formed far out beyond the original one on the side opposite to that from which the littoral current comes, showing that the laws which control the currents and waves are as unerring in their action as the laws which control the heavenly bodies. To predict results, therefore, where the controlling forces are known is no more hazardous for the engineer with experience in such matters than for the astronomer to predict an eclipse.

A comparison of the last chart of the Brazos with one of the Sulina mouth of the Danube at a similar stage of progress of the jetties there, furnishes a strong inductive proof that when the Brazos jetties shall have been built up throughout and consolidated and strengthened so as to effectually resist the action of storms, there will be no more uncertainty about the permanency of the depth there than there is about the permanency of the depth at the Sulina mouth of the Danube or at South Pass.

The author's general conclusions give evidence of his having given the subject of harbor improvements in general a careful and intelligent study. These remarks on the improvement of tidal harbors, however, would seem to require some modification to make them applicable to all cases. He says:

"With tidal harbors, however, the slope of the surface in the jetty channel is inversely as the length of the pass; and consequently on the Gulf where the tides are very small (about 1 foot) and the distance to deep water very great, the plan of improving harbor entrances by confining tidal currents between jetties is a somewhat doubtful experiment."

It is admitted that such a plan is an expensive and unscientific method of improving tidal harbors, inasmuch as the improvement which can be effected by such means, where possible, will be only a fraction of what is possible by other means, and where the form of the tidal curve is that of a sinusoid like that on the Atlantic coast, the experiment is exceedingly doubtful. However, where the form of the tidal curve is such as is developed on the Texas coast during great declinational tides, it is possible by confining tidal currents between jetties to greatly increase the water slope, and hence the scouring force due to tidal action over its normal amount, especially during ebb currents. The results of an investigation made more than a year ago by the writer on water slopes and other tidal phenomena, seem to sustain the position here taken. They are given below, and are entitled—

TIDAL PHENOMENA OF GALVESTON HARBOR.

"For the purpose of investigation we have selected the period of great declinational tides extending from December 20th to 27th, 1889.

This was a time of freedom from wind disturbance. It was also a time of conjunction of the sun and moon, and hence at a time when large fluctuations should be expected. In Fig. 1, Plate CII is given the tidal curve at Red Fish (south side) 17 miles from Galveston Bar and the curve of slope between the bar and Red Fish. Fig. 2 gives the tidal curve at Government wharf, Galveston, 6¼ miles from the bar. Fig. 3 gives the tidal curve at Galveston Bar and the slope curve from the bar to the Government wharf. For the period considered the plane of average level of the water is shown by a full line. The plane of average high water, average low water and the plane midway between them are also shown

TABLE No. 1.

TIDAL PHENOMENA, GALVESTON HARBOR, DECEMBER 20TH TO 27TH, 1889.

Averages.	Bar. Feet	Government Wharf. Feet.	Red Fish. Feet.
Water level	2.90	2.87	2.84
Mid plane	2.57	2.64	2.61
Difference	0.33	0.23	0.23
High water	4.41	3.59	3.23
Low water	0.74	1.69	1.99
Fluctuation	3.67	1.90	1.24

by full lines. Table No. 1 gives the results shown on the drawing, in figures, and also the fluctuations. It was constructed as follows:

The average water level was determined from the plotted curves by means of the planimeter. The slight difference obtained at the different points is probably due to an error in the plane of reference which was determined by water level comparison. The other quantities in the table were determined in the usual way.

TABLE No. 2.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN GOVERNMENT WHARF AND BAR, DECEMBER 20TH TO 27TH, 1889.

DATES.	EBB.			FLOOD.			Ratio of Ave. Ebb to Ave. Flood.
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.	
20th and 21st	8	0.08	0.90	16	0.38	0.80	1.8
21st and 22d.	10	0.75	1.15	15	0.45	1.00	1.7
22d and 23d..	8	0.73	1.50	16	0.42	.90	1.8
23d and 24th.	11	0.73	1.20	14	0.52	1.10	1.4
24th and 25th	11	0.63	1.45	14	0.41	1.15	1.5
25th and 26th	11	0.50	1.25	9	0.47	0.85	1.1
26th and 27th	8	0.31	1.05	15	0.36	0.90	.9
Totals	67	4.33	8.50	99	3.01	6.70	10.2
Means.....	9.4	0.62	1.21	14	0.43	.96	1.5

The ratio of flood to ebb duration is $\frac{14}{9.4} = 1.5$.

TABLE No. 3.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN BAR AND RED FISH SOUTH, DECEMBER 20TH TO 26TH, 1889.

DATES.	EBB.			FLOOD.			Ratio of Ave. Ebb to Ave. Flood.
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.	
20th	3	0.75*	.....	.....	.....	1.55	
21st	9	0.99	1.80	16	0.68	1.90	1.4
22d	10	1.23	2.35	15	0.85	1.70	1.4
23d	8	1.19	1.85	16	0.78	2.20	1.5
24th	10	1.32	2.45	16	0.98	1.45	1.3
25th	9	1.02	1.85	16	0.74	1.75	1.4
26th	6	0.87*	1.25	16	0.65	.....	1.3
Totals	55	6.48	11.55	95	4.68	10.55	8.3
Means	$9\frac{1}{6}$	1.08	1.92	15.8	0.78	1.76	1.4

* These are combined and give slope of 0.83 for 9 hours.

The ratio of flood to ebb duration is $\frac{95}{55} = 1.73$.

Tables Nos. 2 and 3 give the average slopes and corresponding durations, together with the maximum slopes and the ratio of the flood slopes to the ebb slopes. The average ratio of duration of flood slope to ebb slope is also given and the maximum ebb and flood slopes. For example: in Table No. 2, December 20th and 21st, the ebb slope between the Government wharf and bar continued for eight hours, the average of which taken at hourly intervals was 0.68 of a foot. The flood slope continued for sixteen hours with an average of 0.38 of a foot and the ratio of ebb to flood of 1.8 indicates that the average ebb slope exceeded the average flood slope by 80 per cent.

By a further inspection of Table No. 2 it will be seen that the average duration of ebb slope between the bar and Government wharf is 9.4 hours, the average ebb slope is .62 of a foot, the average duration of flood slope is fourteen hours and the flood slope is .43 of a foot. The ratio of the ebb slope to the flood slope is $\frac{.62}{.43} = 1.5$. In other words, the average ebb slope exceeds the average flood slope by 50 per cent. Similarly we find the ratio of the duration of flood to ebb $= \frac{14}{9.4} = 1.5$. In other words, the duration of the flood slope is 50 per cent. greater than that of ebb slope.

The reason for these phenomena is apparent from an inspection of the form of the tidal curve. The longer duration of high water shows that

the average level of the water is above the plane midway between the planes of mean high and mean low water, and hence the distance from average water level to low water is greater than to high water. These facts are shown in their relative values in Table No. 1, where it will be seen that the plane of average water level at Government wharf is 2.87 feet above an assumed datum, while the mid plane is but 2.64 feet above the same datum, giving a difference of .23 of a foot. This difference is .33 of a foot at the bar, and .23 of a foot at Red Fish. It will also be seen that the plane of average low water at the bar is 2.13 feet below the plane of average level at Government wharf and the plane of average high water at the bar is 1.54 feet above the plane of average level at Government wharf, giving an average slope of .59 of a foot in favor of the ebb, or a difference of 38 per cent. in favor of the ebb.

These quantities are obtained as follows: $2.87 - 0.74 = 2.13$; $4.41 - 2.87 = 1.54$; $2.13 - 1.54 = 0.59$; $\frac{.59}{1.54} = .38 = 38$ per cent.

Referring again to Table No. 2, it will be seen that the average of the maximum ebb slopes is 1.21 feet, while that for flood slopes is .96 of a foot—an excess of 26 per cent. in favor of the ebb. Also the maximum ebb slope is 1.50 feet, while the maximum flood slope is 1.15—an excess of 30½ per cent. in favor of the ebb. It will thus be seen that the mean ebb slope, the mean maximum ebb slope and the maximum ebb slope are greater than the corresponding flood slopes, and by amounts of 50 per cent., 26 per cent. and 30½ per cent. respectively.

Whether the ebb and flood velocities bear the same relation to each other as the slopes, may be a matter of some interest, although as far as actual scouring force is concerned, that force depends quite as much if not more upon water slope than upon velocity. This subject, however, has received some attention, and in Fig. 4, Plate CII is shown a record of velocity observations obtained at Bolivar Point in 1880, by means of an automatic current meter described in the report of the Chief of Engineers, 1880, page 1220. This velocity curve indicates that the difference in ebb and flood velocities is even greater than the difference in slopes, and a velocity record obtained at a point immediately upon the bar for a shorter period of time indicates the same fact. Nor should such a result be unexpected, for by an examination of the tidal and slope curves it will be seen that to inaugurate a flood current requires that the momentum due to a strong ebb current and the inertia of the water in the bay be overcome, while to inaugurate an ebb current requires only that the momentum due to a very gentle current and the inertia of the water be overcome. We thus see that the ratio of ebb to flood velocities is still greater than the ratio of their slopes.

TABLE No. 4.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN GOVERNMENT WHARF AND BAR, ASSUMING WATER IN BAY TO REMAIN AT AVERAGE WATER LEVEL, DECEMBER 20TH TO 27TH, 1889.

DATES.	EBB.			FLOOD.		
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.
20th and 21st	9	1.38	2.17	14	0.31	0.53
21st and 22d	9	1.75	2.82	15	0.95	1.83
22d and 23d	9	1.31	2.17	16	0.70	1.43
23d and 24th	9	1.51	2.47	16	1.19	2.18
24th and 25th	9	1.20	1.92	16	0.90	1.53
25th and 26th	10	1.17	1.97	15	0.58	1.38
26th and 27th	12	0.75	1.52	13	0.47	1.13
Totals	67	9.07	15.04	105	5.10	10.01
Means	9.6	1.29	2.15	15	0.73	1.43

Table No. 4 has been prepared to show what water slopes would result if the entrance were contracted by jetties to such an extent that the level of the bay would be practically unaffected by tidal fluctuations in the Gulf. By reference to this table it will be seen that the average ebb slope would be 1.29 feet and would continue 9.6 hours. This is an increase in slope above what at present exists of $1.29 - .62 = .67$ feet $= 108$ per cent., and in duration of $9.6 - 9.4 = .2 = 2$ per cent.

The mean of the maximum ebb slopes would be 2.15 feet, which is an increase over what at present exists of $2.15 - 1.21 = .94$ feet $= 78$ per cent.

The average flood slope would be .73 feet and would continue 15 hours. This is an increase in slope over what at present exists of $.73 - .43 = .30$ feet $= 70$ per cent., and in duration of $15 - 14 = 1$ hour $= 7$ per cent. It will thus be seen that any contraction of the entrance by jetties will increase the water slope both during the ebb and flood, and the ratio of increase is greater in case of the ebb. This increase of slope continues until the maximum contraction is reached and the bay is practically unaffected by the fluctuation of the Gulf.

It must be remembered that the increased slopes which would result from contraction of the entrance have been compared with the slopes which now exist between the Government wharf and bar. It will be seen that the slope from the gorge to the bar would be increased by a much greater amount.

SUMMARY OF RESULTS RELATING TO GOVERNMENT WHARF AND BAR.

First.—The elevation of the average water level above the mid-plane at the bar is .33 of a foot, and at Government wharf is .23 of a foot.

Second.—The average water level above the average low water is 2.13 feet, and below high water 1.54 feet—a difference of .59 of a foot, equal to 38 per cent. in favor of the ebb.

Third.—The average ebb slope exceeds the flood by 50 per cent.

Fourth.—The average duration of the flood slope exceeds the ebb by 50 per cent.

Fifth.—The mean of the maximum ebb slopes exceeds the mean of the maximum flood slopes by 26 per cent.

Sixth.—The maximum ebb slope exceeds the maximum flood slope by 30½ per cent.

Seventh.—The ebb velocities appear to exceed the flood velocities in a greater ratio than the ebb slopes exceed the flood slopes.

Eighth.—Contraction of the entrance will increase the ebb slopes, and in the case of extreme contraction, to the extent of 108 per cent. The duration would also be increased to the extent of 2 per cent.

Ninth.—The mean of the maximum ebb slopes would be increased 78 per cent. and the maximum ebb slope would be increased 65 per cent.

Tenth.—The average flood slope would be increased 70 per cent. and the duration 7 per cent.

Eleventh.—The mean of the maximum flood slopes would be increased 49 per cent. and the maximum flood slope would be increased 90 per cent.

The conclusion is irresistible, that the building of jetties at Galveston Harbor 7 000 feet apart, with the expectation that the scouring force developed between them would be about as great as it would be were they placed half that distance apart,* must be an error in judgment.

A. E. Kastl, M. Am. Soc. C. E.—It is proper to state here that from the inception of the work by the Brazos River Channel and Dock Company the plane of reference used is the plane of average flood tide of the Gulf of Mexico, which is 1.2 feet above the plane of average low tide.

In the latter part of April, 1891, the writer was appointed engineer to take charge of the completion of the jetties, including the building of the concrete coping walls on those portions mostly exposed to wave action. At the time, it was understood that Mr. Wisner would be the consulting engineer when the concrete coping walls would be started. It was the intention to carry on the work to completion according to the plans set forth by Mr. Wisner, and it was confidently expected that before the next winter's storms set in the jetties would be built up to the water surface, and the worst exposed portions protected by concrete coping walls. In brief, the plans were as follows: Mattresses were to be placed wherever possible and small breaks filled with loose brush—all to be well ballasted with stone; no mattress or brush to be put in place unless an ample supply of stone was on hand to secure the work. Wherever the work reached the water surface it was to be finished with a layer of large stones (one ton and over), and interstices well filled with smaller

* *Vide* report of Chief of Engineers for 1886, Part II, page 1307.

stones. As the old work was sufficiently settled, and as the new work would settle comparatively little, it was the intention to follow up with the concrete coping walls as soon as possible after the crown work was completed. Accordingly work was started on the plans outlined above, and continued until the latter part of May, 1891. The mattresses were built on ways on the river bank or on barges, as most convenient, and then floated into place. Small stone was placed by hand and large stone by means of a steam derrick mounted on a barge. Subsequent events gave strong evidence that these methods of construction were the correct ones.

Some time during the latter part of May, and during the absence of the writer on leave, one of the receivers of the company, and who had been the original contractor,* arrived and assumed charge of the construction. The orders of the writer were countermanded and the plans of the engineers no longer followed. After this, loose brush was thrown in promiscuously and small stone used for ballast. The large stone was unloaded hurriedly and not placed as carefully as it should have been. The writer strongly protested against this method of doing the work, but his advice was disregarded. At this time the writer was transferred to other work and thereafter had nothing more to do with the work on the jetties, which was carried on without the supervision of an engineer.

In July, the writer was informed that a contract had been let for about 2 000 linear feet of concrete coping wall. Subsequently it was given out by the contractors that this would be built on the east jetty from Station 30 to Station 50. The end of the east jetty is at Station 54. In August, 1891, it having become evident that the concrete coping walls, for the construction of which the writer had been engaged, would not be built according to the original plans proposed by Mr. Wisner, and as there were no proper tests being made of the materials to be used, the writer no longer wished to have his name as chief engineer identified with the work, and therefore resigned his position.

About the end of May, 1891, a few weeks after a rise in the river, the least central depth in the channel over the bar was 14.9 feet, opposite and near the end of the east jetty. The distance between 16-foot contours was 275 feet; between 18-foot contours, 500 feet. The mean distance of the outer 20-foot contour from a line drawn across the end of the jetties was 2 320 feet. The area over which measurements were taken extends from a line 300 feet west of and parallel to the west jetty to a similar line 609 feet east of the east jetty. The inner 20-foot contour was 850 feet above the end of the jetties. The channel did not cut out during the rise in the river, but, on the contrary, it shoaled somewhat, and the bar was pushed seaward. The deepening in the channel occurred after the rise and when the velocity of the current was decreas-

* See Mr. Wisner's paper.

ing, but in the meantime the worst breaks in the jetties had been closed, and, consequently, most of the river discharge was forced over the bar. Had the jetties been in as good condition before the rise, the greater river discharge due to the rise would have cut out an 18-foot channel over the bar and carried out the sediment to deeper water; and as at the time there was a strong littoral current to the westward, the sediment would not have been deposited in front of the jetties.

During the first week in July, 1891, there was a heavy northeast storm. When the storm was at its height the tide had risen to a point over 3 feet above average flood tide. Afterwards, a careful examination showed that almost all the loose brush work ballasted with small stones was swept off the crown of the jetties. The writer had always objected to this method of construction, knowing it to be only a temporary expedient and not calculated to withstand heavy wave action. Experience at the Tampico jetties, where the writer had been stationed the previous year, showed conclusively that the crown of a jetty exposed to storms was never safe unless finished off with large stone. Those portions of the Brazos jetties which had been finished off with large stone suffered very little damage.

About the middle of July, after a period of low river, the least central depth in the channel over the bar was 15.5 feet, opposite the end of the east jetty. The distance between 16-foot contours was 150 feet; between 18-foot contours 550 feet. The mean distance of the outer 20-foot contour, from a line drawn across the end of the jetties, determined as before, was 1 780 feet. The inner 20-foot contour was 1 000 feet above the end of the jetties.

A comparison of the results of the two foregoing examinations shows the following: A deepening of the channel over the bar; a decrease in the distance between the 16-foot contours of 125 feet; a slight increase in the distance between the 18-foot contours; a mean recession of the outer 20-foot contour of 540 feet; and a recession of the inner 20-foot contour of 150 feet. The following table shows the recession of the outer contours along the three lines indicated, distances being measured from a line across the end of the jetties.

Contour.	On the Axis of the West Jetty produced.		Recession.	On a line midway between the jetties produced.		Recession.	On the Axis of the East Jetty produced.		Recession.
	May.	July.		May.	July.		May.	July.	
Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
12	1 830	1 630	200	1 370	900	470	End of jetty.		
14	2 010	1 680	330	1 560	1 190	370	End of jetty.		
16	2 230	1 750	480	1 870	1 580	290	100	0	100
18	2 490	1 940	550	2 170	1 830	340	1 150	1 040	110
20	2 730	2 220	510	2 760	2 210	550	2 250	1 750	500

A study of the foregoing table would indicate that the wearing away of the outer face of the bar was evidently due to the action of the westerly gulf currents which prevailed during the time.

The estimate of brush, rock and concrete made in March, 1891, for the completion of the jetties, was as follows :

Brush for both jetties	5 000 cords.
Rock for west jetty, large and small for ballast	4 000 tons.
Rock for west jetty, very large for end	1 000 "
Rock for east jetty, large and small, mostly large	2 500 "
Rock for east jetty, very large for end	1 000 "
Concrete for west jetty	1 000 lineal feet.
Concrete for east jetty	3 000 "

However, it is proper to state that on account of a heavy storm after this estimate was made, the amount of brush and rock would have been increased if re-estimated.

The amount of brush and rock used up to August, 1891, was about as follows :

Brush.....6 500 cords, at $1.50 on barges.
Rock.....9 800 tons, at $3.60, on barges at the works.

No concrete work was done. All the brush was not used in the jetties, part of it being used in wing-dams ; how much the writer is not informed.

About the middle of August, 1891, an examination showed that on the whole the jetties were in better condition than during the previous spring, with the exception of the outer 400 feet of both jetties, which seemed to have deteriorated considerably. Although up to August, 1891, not as much had been accomplished as should have been, still to the work done must be ascribed the improvement in the channel, permitting vessels of 15-foot draught to safely enter the harbor. That better results would have followed if the plans of the engineers had been adhered to and they allowed to control the work, can hardly be questioned. Much work still remains to be done before the jetties can be considered safe from storms. They should be brought up to the water surface as soon as practicable, and wherever exposed to wave action finished off with a layer of large stone (1 ton and over) carefully placed. Then the concrete coping walls should be built on at least the last 3 000 feet of the east jetty and the last 2 000 feet of the west jetty. Until the jetties are so finished there will always be a tendency for the bar to advance with every rise in the river. Then during seasons of low river the action of the sea waves and currents on the outer face of the bar will tend to shoal up the channel at the entrance to the jetties. Whenever the jetties are completed on the plans outlined the next rise in the river will create a 20-foot channel over the bar. The bar material will be

carried out a considerable distance beyond the 20-foot contour, and if the usual conditions prevail, no deposit will take place in front of the jetties. The writer is of the opinion that it will not be necessary to extend the jetties beyond their present length in order to secure and maintain a 20-foot channel. This, however, is conditioned on the thorough completion of the present jetties before the next heavy rise in the river. However, if the works are allowed to deteriorate, a succession of river floods will so advance the bar that it will eventually become necessary to extend the jetties.

The writer is entirely in accord with the principles set forth by Mr. Wisner at the conclusion of his paper, particularly the seventh.

R. E. McMath, M. Am. Soc. C. E.—The history of the improvement at the mouth of the Brazos sets in violent contrast the official and private methods of conducting such an enterprise. A study of the paper shows manifestly that the difference is in the methods and not in the men. The Government engineer is handicapped by restrictions and regulations which leave him little freedom of action, everything must be referred to a distant superior. The whole history enforces the idea that work of this kind should be entrusted to an experienced engineer, who is on the ground taking immediate and responsible charge of the work; one who is observant, quick, and ready in resource, bold to go forward and free to act according to his best judgment.

I therefore heartily concur with the author in the statement made in the seventh and last paragraph in his summary. I can also, from experience, indorse the fifth conclusion, viz., scouring action works up stream.

J. F. Le Baron, M. Am. Soc. C. E.—The abundant success achieved at the Brazos River mouth is peculiarly gratifying to the civil engineers of the country. There is nothing that succeeds like success, and success has been so marked in this case, and the plans so judiciously and wisely designed and ably carried out, that it leaves little room for criticism or discussion. There are, however, a few things that occur to me which it might be well to discuss, and for this purpose one might wish that the chart furnished with the paper was a little more complete and contained more soundings of the adjacent portions of the Gulf, together with the submerged contours and locations of borings, with the line of breakers and some general facts in regard to the force and heights of waves observed there; also of the widths of the mattresses used.

In the sixth conclusion arrived at by Mr. Wisner in his very interesting and instructive paper, he says that the action of the currents and deposits when the east jetty was extended 2 000 feet beyond the west jetty, "very plainly indicate that one jetty would not be very effective for channel making at ports of this class." This assertion may be considered a little too sweeping, and leads us up to the inquiry if a different plan might not have been adopted in this case that would have dispensed

with a portion at least of the west jetty, and so have effected a considerable saving in cost.

It is a well known fact that the concave bends of rivers and training walls always induce a scour and are deeper than other reaches of the river. At Swinemunde, one of the mouths of the Oder River, Prussia, on the Baltic Sea, advantage has been taken of this fact in planning the jetties for the improvement of the channel over the bar. This river is about the same size as the Brazos, with a drainage area of 50 000 square miles, and empties into an almost tideless sea. The prevailing storms come from the northeast, and there is at other times a somewhat strong littoral current to the eastward.

The engineer in planning the east jetty gave it a radius of about 5 500 feet. This is evidently too short, the result being that too much scour has been developed alongside this jetty and deeper water produced than was needed. The east jetty projects about a quarter of a mile beyond the west jetty, which is built nearly parallel to it. The littoral current strikes the concave side of the east jetty in this case, which would be considered unfavorable, but the river current proves strong enough to sweep away all its sediment.

In the case of the Brazos River the littoral current would strike the convex side of the east jetty, provided it had been built on a curve concave to the west. There is another advantage in building such jetties on a curve. The waves rolling in at the mouth will expend themselves on the concave jetty, thus insuring tranquil water in the larger part of the channel instead of rolling up the whole length of the channel to break on the shore. The construction of the windward jetty on a curve entails no increased expense, and if the requisite depth of water is not obtained with one jetty the other can be constructed parallel to it, and in the interests of economy or when quick results are desired to demonstrate the feasibility of obtaining a channel, to a faltering company it seems worthy of trial. Other examples of curved jetties might be cited.

In regard to the width between the jetties, this seems to have been judiciously chosen. It is fortunate that the recommendation of Sir John Coode, of only 350 feet width between the jetties, was not adopted, as in the result of excessive scour and undermining of the jetties, which would undoubtedly have taken place, no room would have been left for building spur walls for protection (as has had to be done even with the adopted width of 600 feet) without fatally contracting the available fairway for vessels. In all such cases I believe in giving a little more width than the result obtained by calculation, as, if found too wide, the channel can readily be contracted by building spurs, which will cause an accumulation of sand, etc., and greatly strengthen the jetty and protect it from the teredo ; but if the fatal mistake is made of building them too narrow, we are left almost at the mercy of the currents.

From the reports of the Chief of Engineers for 1875–79 and 1881, it

appears that the mouth of the Brazos is an exposed place and subject to heavy seas on the bar. The party sent to survey it waited over two weeks for the bar to be smooth enough to allow of venturing out in a boat, and it was then obliged to postpone the survey until a more favorable time. This fact and the fact that the brush mattress jetties built by the army engineers were destroyed show that the waves act powerfully at this place, and that jetties, to withstand their action, must be strong and solidly built. The question arises, if brush and stone jetties will prove strong enough here. In fact it would seem almost the height of imprudence to have used the same class of construction that so signally failed when employed by the Government engineers. After all, it is only a question of stone. If sufficient stone, and large enough, can be piled on the mattresses and kept there, there is no doubt that they will stand; but for several reasons I should prefer in such an exposed position to use log mattresses loaded with stone, such as have been successfully used at Charleston, Cumberland Sound and St. John's River jetties. I was engaged on the last two named works three years from their commencement as principal United States Assistant Engineer.

Both of the places are exposed to the full sweep of the Atlantic Ocean, as the mouth of the Brazos is to that of the Gulf of Mexico. At both Cumberland Sound and the mouth of the St. John's River the object of the improvements is to confine the waters of the Sound and river by parallel jetties where they cross the bar, and so produce a greater depth of water by the induced scour. The bars and headlands at both points are composed of sand. The details of construction have been almost identical in both places. The jetties consist of a foundation course of mattresses from 20 to 87 feet in width, and later up to 120 feet in width, which are sunk on the jetty range and loaded with broken rock to a depth of 12 inches. Successive courses of mats, each 8 feet narrower, are sunk, and each covered with a foot of stone, until the work is brought up to low water. These mattresses were designed by the late General Q. A. Gillmore, M. Am. Soc. C. E. They consist simply of a raft of logs, not less than 9 inches at the small end, which are held together by binders of logs not less than 4 inches at the small end, placed 8 feet apart and spiked to the logs below by 18 or 20-inch spikes. On some of the mats a layer of brush 9 feet wide was placed on the edges, projecting over the sides of the mat, and intended to be 5 inches thick when compressed by the stone. On others the brush covered the whole mat. Poles and sawmill slabs were substituted for the brush in some cases. The brush, especially the 9-foot layer on the sides, was washed off by the waves in a short time. The thickness of the mats was from 16 inches to 22 inches. These mats were very easily made, and required no skilled workmen. No ways were required. The logs were floated down to the log camps, situated 10 to 15 miles up the river, where a shallow shelving beach occurred, and pushed up together, being afloat and held in place by

cant hooks while being spiked to the binders. A well braced mast was set up in the center at each end, the slabs or poles spiked on parallel to the logs, or the brush, when used, bound on with manilla rope, and the mattress was complete.

Seven men would make a mat 100 × 100 feet in three days. The lowest contract price under which these mats were made was 45 cents per square yard. The price varied with different contracts. My estimate of the cost was 35 cents per square yard. Stone was brought out in schooners from New York City and New London, Conn., as ballast, for $1.50 to $1.75 per cubic yard on the vessel's rail. Lime rock from Florida cost $2.45 per ton on the works. It requires some little skill to sink these mats, especially in rough water. This part of the work should be intrusted to good sailor men. The mats are towed out to the works by a powerful steam tug. On each side of the mat is a large, strongly braced decked lighter carrying the stone. Large ship's anchors are previously placed in position, one on each side of the jetty, and buoyed, and the lighters are made fast to these anchors by lines around the capstans, and the mat is hauled up abutting on the last one laid, its position being determined by the masts.

The mat is ranged by its masts, with ranges on shore. During this time the mat is held up by bridle lines at the corners. Stone is now thrown on and the lines slacked evenly until the mat gradually sinks to the bottom. As many as three mats were sometimes sunk at one time. Low water slack is chosen for setting the mats in deep water, and high water slack on the shoals. It is astonishing how accurately mats can be set in calm water by experienced men. But when the tide turns and sets strong before the mat is sunk, it often happened that gaps of 4, 8 or 12 feet would occur, or the mat be as much as this out of line. The plan was therefore adopted of lapping the mats 6 to 8 feet. Sixteen men would set a mat in two and a half to three hours, but in the breakers it took double that time. It was found that the teredo did very little harm to the work. The foundation course was almost invariably sanded over, sometimes as much as 10 feet deep. In the upper courses it was found that the worm ate the exposed ends of the logs and the outer binders, but did not penetrate into that part of the log that was covered with rock. At Cumberland Sound and at St. John's River bar the rock was in a short time completely covered, and all the interstices filled with a luxuriant growth of different species of shell fish—oysters, barnacles, and various bivalves. This effectually protected the logs from the teredo, and induced the deposit of silting sand.

The south jetty at St. John's bar was at one time in imminent danger of being totally wrecked. This jetty crossed the sailing line, as it then existed, and a gap was left for the passage of vessels. The jetty on the shore side of the gap was brought up to low water-line nearly, by about six courses of mattresses, the water being about 22 feet deep. The

result was that the overfall of the current sweeping over the jetty produced eddies and whirlpools that gullied out holes close to the jetty, in some cases over 50 feet deep. The condition of the work then was that of a wall about 20 feet high, standing on top of a sand embankment of about the same width and 9 to 25 feet high. The results were what might have been expected. The currents at once cut through under the jetty, washing away the sand embankment, and sections of the work settled into the gaps thus formed. In one case four courses of the jetty tumbled in, the mattresses tilted until they discharged their layer of stone and the logs floated off, the stone sinking into the live sand and disappearing completely. To arrest this action eight spurs were built, four on each side. These not having the desired effect, an apron 100 feet wide was laid on each side of the work. It consisted of the regular mattresses loaded with stone. This had the desired effect and arrested the scour. The damage was greatly augmented by the works being left without an appropriation for over a year. This system of piecemeal appropriations is most costly and pernicious in works of this kind, and a worse could not easily be devised.

The action just described was observed also at Cumberland Sound, but in a lesser degree, and was checked in the same manner. The jetties at both places were planned with too narrow foundations. Fifty feet was the ordinary width in shoal water, but it was found necessary to increase this to 100 feet, and in the breakers to 120 feet.

The velocities of the currents on the St. John's bar were determined by the U. S. Coast Survey as follows, in feet per second, and are given on its chart:

Ebb,	1st	quarter,	2.03.	Flood,	1st	quarter,	0.84.
"	2d	"	3.33.	"	2d	"	1.66.
"	3d	"	1.15.	"	3d	"	0.84.

The foregoing facts show the necessity of constant watchfulness in building works of this kind, and the impossibility of making an iron-clad plan to be undeviatingly followed. They also show the necessity of wide foundations and aprons. The question of log or brush mattressess must be decided for each locality. In some places brush is very plentiful and logs are scarce, but where logs can be obtained at reasonable expense I consider them preferable to brush for mattresses, for the following reasons:

First.—Greater solidity, strength and simplicity.

Second.—Greater facility in making the mattresses, allowing greater speed and less expense.

Third.—No ways are required to make them on, or false works of any kind, and men do not have to be educated in their manufacture.

Fourth.—A jetty composed of logs and stone affords a firmer base than brush, for a stone or concrete superstructure.

The method of making and sinking the mattresses at the Brazos was certainly very ingenious, but the false works and construction of the mattresses under them could hardly be prosecuted in a line of breakers; certainly not in such a sea as we worked in at Cumberland Sound and at St. John's bar. The expense of the false works, track and cars must certainly be considerable ; and it seems, from Mr. Wisner's paper, that there were also required a tug and lighters ; so I cannot see any advantage on the score of economy. Should an extension of the jetties be found necessary, or breaks occur, the trestle and track must be rebuilt the whole distance, or recourse had to tugs and lighters ; and these must be furnished any way for laying apron and spurs. For the reasons stated above I am strongly in favor of sinking mats on ranges, in preference to using a trestle.

From my experience in sea works I am strongly adverse to what is commonly called fine workmanship, or fine joints and carpenter work, or any construction that depends on iron fastenings under water. Only the roughest Titan-like methods will withstand the constant tremors and shocks of the waves. The fine cabinet work executed in making gabions at Galveston is an example. These carefully made cylinders were broken up and destroyed in a single season.

On the Nicaragua Ship Canal (of which I was in local charge in 1888 and 1889), the construction of the breakwater for protecting the channel entrance at San Juan del Norte—the eastern end of the canal—was commenced by building a pile wharf, the bays being filled with brush and broken stone. The outside edges of the wharf were piled close with creosoted piles. The inner piles were uncreosoted. The width of the entire structure was only 40 feet, and the piles were sunk by the water jet, aided by steam pile drivers. The plans of this somewhat novel structure for such a position were sent out from the States. The depth of water was 6 to 18 feet, and it was exposed to the full sweep of the sea. As was to be expected, the teredo speedily found its way between the creosoted piles forming the outer row, and attacked the uncreosoted interior piles. The waves breaking over this frail structure began undermining it, but the shore end was providentially saved by completely sanding over; the foreshore, to the eastward, as had been predicted, advancing rapidly and filling up the windward angle with sand.

In the further extension of this breakwater it is proposed to use stone alone, but rapidity of construction and economy will be secured by using log mattresses as recommended in my design and report on the harbor. The uncreosoted piles for this work cost from $6 to $8 apiece, landed from the ship, before being driven. A log mattress and stone jetty, as originally recommended, would have been far more economical and permanent. A great abundance of black palm logs, impervious to the teredo, besides other suitable timber, exist within half a mile of the work. This work teaches the same lesson : avoidance of carpenter-work

and the necessity of most careful and constant espionage of the work as it progresses, with abundant and ready means at hand for laying aprons for protection from scour.

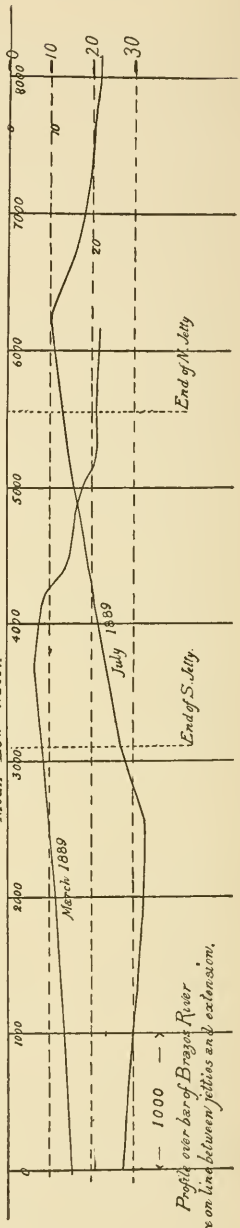
The experiences at the Brazos River and elsewhere on the Gulf of Mexico emphasize the fact so often stated, that the engineer learns quite as much by the failures of (himself and) others as by their successes; and the result it is hoped will be the adoption of thoroughly rational and successful designs for all improvements hereafter entered upon in these waters or others similarly situated.

W. R. Hutton, M. Am. Soc. C. E.—The discordance of the different descriptions of the physical conditions at the mouth of the Brazos River and its bar do not permit an intelligent discussion of the plan of improvement now in progress. The opinions expressed concerning its capabilities for improvement are equally discordant.

In 1872 and 1875, Major Howell, United States Engineers, recommended converging pile jetties by which the depth in the lower river, 15 to 18 feet, would be increased, a channel of somewhat greater depth carried across the bar, sand would be accumulated behind the jetties, and there would be a slow extension of the bar seaward.

Later, Major Mansfield, also of the army, whose recommendations were approved by a board of engineers, recommends parallel jetties of brush and stone; but as the depth of the river mouth is only 10 feet at its most contracted part—this is the greatest depth that can be maintained over the bar. Although there is a slight deposit in bends, there is none near the mouth, and the river carries no sediment. Jetties will transfer the river mouth to the bar, and we must trust to the littoral current to delay its reformation.

In 1887, Major Ernst, United States Engineers, reported the "proposed" improvement as impracticable and any improvement doubtful. That the depth scoured by the



greatest freshets, which could hardly exceed 10 feet, will be reduced to 6 feet or less by the action of the waves, which tend to form a smooth shore line in gentle curves. This they would soon accomplish here if left to themselves. The river alone prevents them.

Mr. Wisner, whose observations are understood to extend over two or three years, sustains Major Howell's opinions, both as to physical facts and as to results.

There seems to be no question that the discharge of the river during a large portion of the year is very considerable, enough to provide amply for any improvement so far as the element of scour is concerned. What is of great importance, but less exactly stated, is the period and duration of low water and its coincidence with other conditions. Mr. Wisner says it covers from one to three months each "season." Does he mean each year? Is it one to three months consecutively? The whole treatment of the case depends on the length of time during which the littoral movements and normal wave action may operate to injure the full river's work.*

The depth curves drawn upon Mr. Wisner's map of 1889 show the existence of a littoral current, as mentioned by him, which causes the deposits to form to the southwestward of the river mouth, the outer face of the bar being quite steep under their influence. The 20-foot curve is not affected by it, for it converges slightly toward the shore to the south, while the other curves diverge markedly from the shore line. The depth lines on the map of 1890, after the construction of the north or east jetty to a length of nearly 6 000 feet, and of the south or west jetty to more than 3 000 feet, show the 20-foot curve to have moved out in front of the jetties some 2 000 feet; the 10 and 15 foot curves show still greater displacement; and a more accentuated divergence from the shore line and a steeper slope of the outer face of the bar.

It is not to be inferred from the foregoing remarks that in a general way the works proposed will not produce the result desired. A further study of the conditions mentioned is nevertheless desirable, that such modifications or additions to the present plans as may be found necessary to insure their permanency, may be introduced in advance and not left until their need is developed by experience.

In all plans for improvement by jetties the character of the material transported is to be considered. At the mouth of the Mississippi the light character of the material is most clearly indicated by the instability of the land formed by it. At the mouth of the Brazos the bar is described as hard sand—though according to Mr. Wisner, the greater portion of the sediment is of a very light character. This is another evidence of the strong coastwise current which carries off the light material.

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* Those two factors, the travel of the sand along the beach and the action of normal waves, are mentioned, but no idea is given of their magnitude or of the conditions under which they operate.

It is to be regretted that both jetties could not have been carried out rapidly and simultaneously to the depth proposed. The late Captain Eads has most strongly emphasized the necessity of extending them at once beyond the bar, so that further sediment brought down by the stream may be discharged in deep water where the transporting currents are strongest.

Mr. Wisner describes the bar as formed by deposits from river floods and by sand drift moved along the coast by current and wave action; he also says that the outer face of the bar has been worn away under the action of waves and currents. It would be interesting to know the conditions under which the same causes produce such opposite results.

George Y. Wisner, M. Am. Soc. C. E., closing the discussion, writes: Colonel Mendell is mistaken in regard to the alignment of the channel across the Brazos Bar after freshets. A heavy rise in the river always produces a channel curving to the eastward against the littoral current, the same as at the mouths of all rivers flowing directly into an open sea where strong littoral currents exist.

In regard to the low slope of the bottom of the gulf outside of the bar (1 foot in 500), it may be well to remark that it is six times that at Galveston Harbor, which does not seem to produce any fears on the part of the engineer having that work in charge as to the ultimate result.

The question raised by Colonel Mendell as to whether the bar may not reform in front of the jetty entrance is the same as that so strongly adhered to during the construction of the jetties at the South Pass entrance of the Mississippi River. The bar formation to the westward of the South Pass jetties, as has been so well stated by Mr. Ripley, is almost identical in conformation with that at the mouth of the Brazos, and as the currents there for twelve years have produced no indication of the bar reforming in front of the works, there is no reason to fear that any different result will occur at the Brazos. As to whether the littoral currents in the gulf are capable of taking care of the sediment annually carried across the bar by the river, there can be no question, as shown by comparison of the contours of the gulf bottom since 1858. The 20-foot contour is no further seaward in front of the jetty to-day than at that date, and in 1881 was 1 200 feet nearer shore, showing conclusively that the gulf currents are ample to dispose of all material carried by the river. With these facts before us it would seem that Colonel Mendell practically admits that a 20-foot channel is an assured result. Even though the conditions were badly stated in the original paper, it is difficult to infer why he and Colonel Ernst should think that the paper had better have been deferred until final result was obtained; for if such was the case nearly the entire annual report of the Chief of Engineers would be subject to the same criticism, as the harbor improvements there discussed are not likely to be completed for many years.

In regard to the method of mattress construction at harbors on the Pacific coast, reference should have been made to the report of Chief of Engineers for 1890, page 3 022, where the methods now in use there are first described.

It is true, as Mr. Le Baron states, that the concave bends and training walls induce a scour and are deeper than other reaches of the river, but this greater depth is always obtained at the expense of channel width, a very important element at a harbor entrance where strong currents exist. At South Pass the course near the outer end of the jetties has a tendency to make the channel less than the required legal width, and necessitates the keeping of a dredge boat in constant commission to cut away deposits from the convex bend of the channel which invariably occur at certain stages of the river. A curved, narrow channel is very dangerous to enter whenever the current in the river is strong, and such construction should be avoided, if possible.

As to the strength of brush jetties, there is no question that when properly protected with concrete coping walls they will withstand any sea on the gulf coast, but without such protection destruction is inevitable. The cost of construction of one cubic yard of brush mattress, including all false works and superintendence, as may be seen from the table in the original paper, was only 38 cents, an amount that compares very favorably with that of the log mats used by Mr. Le Baron. It is but fair, however, to state that the cost per cubic yard of the completed brush work in the Brazos jetties was only one-sixth of that previously done there by the Government. The work was conducted without difficulty at all times, except during very heavy storms, without loss of material or danger to the crews. Tugs and lighters were used for the reason that all material was brought down the river in that way, and whenever possible was discharged directly into the works from the barges.

The sand drift moving along the shore, which originally was forced out over the bar during freshets, is now arrested by the east jetty and forced up into a high concave sand beach to the eastward of the works. As before stated, this action does not extend much beyond the 18-foot contour, and consequently will play no part in bar formation beyond the jetty entrance. From this I think it will be seen that the same causes are not expected to produce opposite results.

Mr. E. L. Corthell, M. Am. Soc. C. E., sent in a brief discussion, too late to be forwarded to the author of the paper.

He mentions the use of the system of building and depositing the mats adopted at the Brazos as having been in its essential particulars in previous use at Yakima, Oregon, and at other points on the Pacific Coast.

It was adopted at Tampico on works under Mr. Corthell's charge. At Tampico piles were used instead of scantling for grillage, and

"outriggers" were used to give additional width to the mattresses. The last feature was adopted for the purpose of making a much broader base than could be obtained otherwise, except by giving great width to the trestle-work; and it may be stated that by means of these "outriggers" mattresses 80 feet in width, 5 feet in thickness and 60 feet in length have been built at Tampico. These "outriggers," or overhangers, are generally 14 feet in length, built of planks, bolted together; they rest on the end of the caps of the trestles, overlapping the caps to the stringers of the track, and overhanging the ends of the caps, generally, about 9 feet. They are fastened to the caps with chains made for the purpose. There were conditions existing at Tampico which compelled also some different methods than were necessary at the mouth of the Brazos, the former place being much more exposed to the sea; and the work was carried on by means of an ordinary double track railroad trestle, with from six to eight piles in a bent, the center piles generally being creosoted and the brush and stone hauled out over the trestle by an ordinary locomotive and flat cars; although there is considerable work on the south or unexposed jetty, which was done by means of floating equipment, the brush and stone being brought to position alongside the jetty by barges.

Mr. Corthell states that at the South Pass, nearly $1\frac{1}{4}$ yards of heavy limestone was used on an average with every cord of brush, although a much less quantity was used in simply sinking the mattresses.

In reference to the conclusions at the end of the paper, Mr. Corthell agrees with them all most heartily. As to the seventh, that the engineer must have full control of all plans and also of the methods, material and rapidity of construction, he agrees with it in general, but says the experience, character and readiness of the contractor to conform to the specifications and instructions given from time to time by the engineer may modify this conclusion.

As to information of the latest date in reference to the channel at the mouth of the Brazos, he is in receipt of a telegram of date of November 27, 1891, to the effect that a vessel of $16\frac{1}{2}$ feet draught has been taken in from sea, with a tide four-tenths of a foot below average flood. When the works shall have been completed and the 20-foot channel obtained, of which there is no doubt whatever, and a great commercial port established, of which, also, there is no doubt in the opinion of Mr. Corthell, a most interesting engineering and commercial paper might, and probably will, be written, of great interest to the Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

512.

(Vol. XXV.—November, 1891.)

DISCUSSION ON PAPER No. 488—"A MEMOIR ON WATER METERS."

By G. LEVERICH, M. Am. Soc. C. E.

In 1865-66, in levying the United States Internal Revenue tax on whiskey, it was determined that the product should be measured by passing it from each still through a fluid meter carefully enclosed and under lock and seal. To meet the large probable demand for these machines, designs were called for, and in consequence the following was prepared, under date February 26th, 1866, the whole being copied from an old note book, as then recorded.

FLUID METERS.

An automatic measurer of non-elastic fluids, as water, must have a cavity or recess, which, enlarging as the fluid enters and contracting as it passes out, accurately measures the quantity expelled. This variable space may be planned as in two of the accompanying designs, Nos. 2 and 3; consisting of a piston moving backward and forth in a cylinder —appropriate devices opening and closing valves to permit the alternate passing of the fluid and actuated by its weight or pressure; a flexible material may be employed so that the chamber folds up and expands as a bellows; or an elastic fluid, as air, may be used to fill the space left by the water and at a proper time take its place. This last is shown in

design No. 1. Again the movement of the parts may be, as in this design, rotary, and the flow of the liquid be uninterrupted and in one direction, or they may reciprocate either in linear or curved paths, which will cause the flow to be periodic, more or less merged into a continuous one, and the direction will regularly change.

It is desirable in designing a new meter of this class to secure ease and certainty of action, a continuous flow, accurate measurement and simplicity in construction; under all heads it should work well and its cost should be small. Ease and certainty of action are requisite, that light heads may effectually operate the machine, that the wear be slight and that under usual circumstances no stoppage occurs; a continuous flow is desirable, that the pipes be not unduly strained by opening and abruptly closing passages; accurate measurement should be secured by taking care that the parts may not change direction irregularly and that the chambers be uniformly emptied; simplicity of construction is necessary, that the cost of making and of repairing be small and thereby the demand for and use of such machines increase.

With these remarks, or rather suggestions, the designs presented are next to be considered; it being understood that they are crude, and to be modified in proportion and detail as may appear best upon further development and trial of the plans. A careful examination of Patent Office Reports from 1851 to 1862 shows nothing conflicting with their originality upon that record. Of course, in this, as in most similar cases at present, where new designs are made, involving old and much used principles, the originality consists mostly in the arrangement and combination of parts. By way of parenthesis it may be mentioned that Worthington's Patent Water Meter* is the only one in general use.

Design No. 1, as sketched, is shown by Figs. 1 and 2, Fig. 1 being an end view in section through the middle and Fig. 2 a side view, the left half in axial section and the right half a full view. This design consists of a chambered cylinder, made slightly conical and surrounded by a case having an entrance passage on one side and an exit passage on the other, and on the top an air chamber. In the sketch e is the cylinder, having eight chambers or cavities (this number may be changed), m is the case, d the inlet, f the outlet, and a the air chamber connected with the case by the passages b and c. The cylinder is closed

* United States Letters Patent No. 13 320, recorded in United States Patent Office Reports for 1855, Vol. I, 718; Vol. II, 184.

at the ends, as shown at one end in the sectional side view, and revolves upon the journal g and pointed pivot i; the journal being packed by the bushing h, which screws into the case and with the pivot i, allows the cylinder to be adjusted endwise, so that it touches the case along the inside and yet easily revolves. It is thought that no packing—that is, of hemp or leather—will be required, and if the workmanship is of good quality, no provision to prevent leakage will be requisite; the friction of movement being small when the cylinder is properly adjusted. This adjustment should be such that a thin film of water or fluid shall separate the rubbing parts, being held there by capillary attraction. Should it be found otherwise, a system of grooves may be cut, as shown in the sketch,

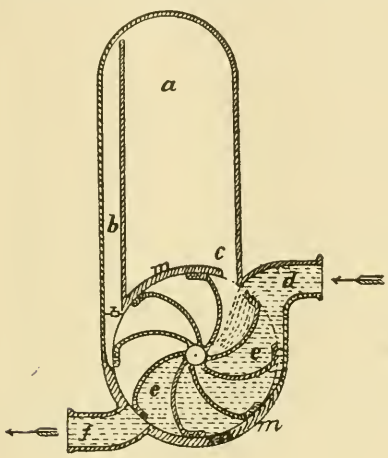

FIG. 1.

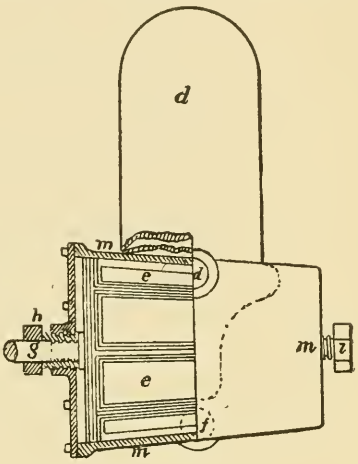

FIG. 2.

which, acting upon the fluid, retard its flow, each similar to a thin plate orifice and furnishing two sharp edges; n being the number of grooves, and f the co-efficient of the head left after passing one, the leakage for n grooves will be expressed by the term $f^n h$. (Similarly this remark upon packing will apply to the other designs wherever grooves are drawn.) It should be stated that this cylinder may be kept adjusted by allowing a slight motion endwise in the journal and pivot, and permitting the fluid to enter the case between its and the cylinder ends; the difference in size of the two ends causing sufficient difference of pressure to keep the cylinder loosely in contact with the case.

The action of this meter is obvious. To the shaft g any suitable registering device may be attached. A simple one has been planned, but

it is not thought worth while to sketch it here for either of the designs. The capacity and height of the air chamber above the case depend upon the pressure of the fluid to be measured, and whenever it is desirable to preserve the pressure head, the proportions should be such that air in the chamber to be filled by water will, when displaced, compress that in the air chamber so that the water line will not be above the entrance at the case into the passage *c*. For measuring fluids under slight pressure the chamber *a* may communicate with the atmosphere.

The chief doubt in reference to the success of this plan is whether the water may not absorb and carry off some of the air enclosed, so that the water line will be above the orifice of *c*, in which case the passage will cease. A scheme has been thought of, to maintain this line constant under variable pressures, and if the air be absorbed. Of course where slight pressure is had, this doubt does not obtain.

It may be noticed that in the sketch, one revolving chamber is filling and none are emptying. The parts should be so proportioned that the filling and emptying be simultaneous.

Before proceeding to the description of the other sketches, attention may be called to the use of this machine as an automatic boiler feeder. Inlet *d* being closed and outlet *f* connected with the boiler at the water line, steam will displace the water in the chambers (revolved by power) only when water in the boiler is below the orifice of *f*. The passage *b* should connect with a water tank and discharge the steam into it through a syphon top (to prevent water flowing down this pipe), and the passage *c* should connect with the bottom of this tank, it being placed above the boiler. It will be noticed that for each measure of water fed into the boiler an equal measure of steam is allowed to escape into the feed water tank, thereby heating the water, and without attention, the water in the boiler may be kept at a constant level.

Design No. 2, Fig. 3, is given to suggest what should be avoided in practice rather than with any notion of its practical utility. An oscillating meter is shown in side view, the right half in axial section and the left half full, with the cover of the surrounding case removed. This meter consists of a cylinder *e*, containing a loose piston *b*, which together oscillate or tilt on the pivot *g*, to which the register is attached; *d d d'* are the inlet and *f f f'* the outlet pipes, having ports as shown, one discharging or receiving while the other is closed by the wings *c c* of the cylinder ; the ports are kept tight by the leather or rubber gaskets

a a, recessed around them and pressed by the fluid tightly against the wings. To prevent tilting before the piston has reached the discharge port, catches are provided to hold the cylinder until released by the piston pressing against them ; these are not drawn.*

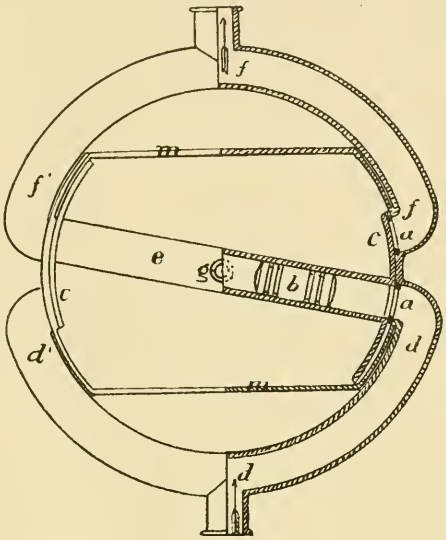

FIG. 3.

Design No. 3, Figs. 4 and 5, is another meter, which is thought well of for water under pressure. Fig. 4 is a side view, the right half in full and the left half with front of case removed. Fig. 5 is a plan of the case, the valves being removed. This meter consists of the case *m*, which is

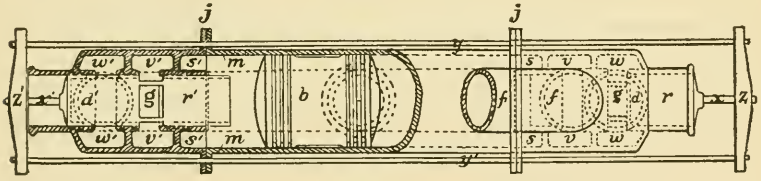

FIG. 4.

a plain cylinder, the inlet pipes, *d d d'* with chambered ports *w* and *w'*, the outlet pipes *f f f'* with chambered ports *v* and *v'*, the loose piston *b* which traverses the cylinder and moves the piston valves *r* and *r'*, said

*Mr. Thompson, the writer of the Paper of which this is a discussion, under date of October 15th, 1891, says : "It may interest you to know, that I only recently had a new meter scheme brought to me, as something which dead sure had solved the problem, that in all essential particulars is identical with that shown in your design No. 2."

valves being hollow and closed at the outer ends, each containing a port s and s' and being connected rigidly together by the rods $x\,x'$, yokes $z\,z'$, and rods y and y'; w and w' are inlet ports, v and v' are outlet ports, chambered, or rather connecting with chambers, which are as the ports annular and reach around the piston valves r and r'; s and s' are recesses containing "dead" fluid and will receive and hold sediment until it is removed. The ports in the valves $g\,g'$ are made somewhat wider than in the case; it will be noticed that the valves are perfectly

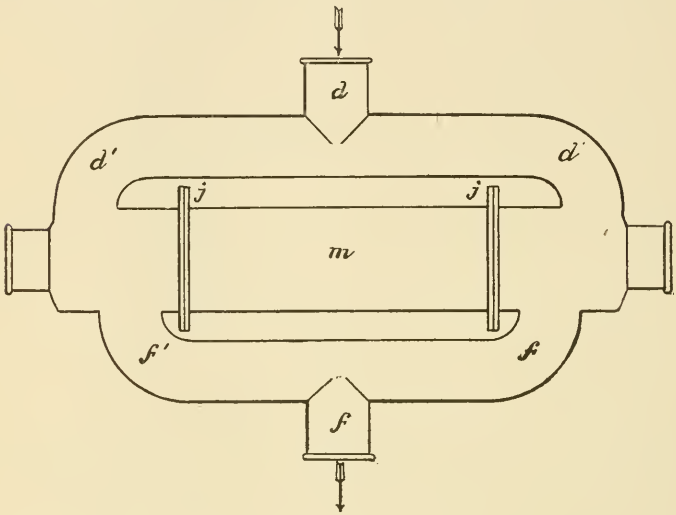

Fig. 5.

balanced, the pressure upon them as they are connected together being nearly equal in every direction. The piston needs no packing, as it moves loosely in the chamber; this and the valves are shown grooved; if it should be necessary, the pistons may be packed with "cup leathers" or otherwise; the case m need not be more than three diameters long between the flanges $j\,j'$. Of course in use, the valve connections should be covered to prevent fraudulent tampering with the register, which may be attached to any convenient place on these connections.

The operation is as follows: Fluid entering through the chamber port w, and piston port g, as shown at the right in the sketch, passes toward the left, carrying the piston b before it; the outflowing fluid passing off through the piston valve r', ports g' and v', into the pipe f' and so outward. When the piston b reaches the valve r', which projects into the

cylinder as shown, it carries the valves to the left, closing the port v', nearly closing the port w, opening the port w' slightly, and leaving the port v ready to open. As soon as the port w' is opened, the fluid enters and equalizes its pressure through the port w in a contrary direction, whence the piston b moves a small space further until brought up against the end of the valve casing, thereby closing port w entirely. The fluid pressure now opens wide w' and v and the flow is reversed, the piston b' moves to the right, the operation is repeated, and thus becomes continuous.

Subsequently to writing the foregoing record and previous to December 8th, 1860, two of these designs, No. 1 and No. 3, were elaborated as shown for design No. 1, by Figs. 6 and 7, 8 and 9, and 10 and 11; and for design No. 3 by Figs. 12 and 13, and 14 and 15. These are to scale and were intended as the preliminary drawings of working machines. It is thought that a detailed description is unnecessary.

DISCUSSION.

JOHN THOMSON, M. Am. Soc. C. E.—The author has read with much interest Mr. Leverich's addition to his little paper on water meters; being particularly impressed with the fact that some twenty-six years ago there evidently was as clearly defined a comprehension of the fundamental principles involved as at the present time, when the actual use of and experience with meters is quite out of comparison with that of the date to which this refers.

Personally the author never had experience with the mechanical measurement of the fluid for which Mr. Leverich's designs were specially adapted; although he came rather near to this not long ago, when a well known member of our Society placed an order with his company for a "Cocktail" size; which order, by the way, it was proposed to fill with a meter of 6 inches capacity having an eel trap attached. But seriously, there is a very material difference in the difficulties to be met in measuring fluids of different qualities. Possibly the term "viscosity" would better express the author's present meaning. Thus a meter which may measure water accurately will require special calibrations for oils of different grades, because of the adhesion of the oil to the moving parts, which thus affects the displacement of the measuring mechanism.

In regard to the grooves for preventing leakage past the surfaces of the cylinder, as in Fig. 2, the author has never before seen this feature

more clearly expressed, and believes that the value of this detail in such constructions is not fully appreciated both for the particular function intended by Mr. Leverich and also as a means for relieving such actions when obstructed by the introduction of fine particles of sand. In a recent experiment upon this very detail, using a freely fitted plug, first tried smooth and then grooved, with a difference of pressure equal to about 20 pounds to the square inch, the leakage past the plug when grooved was, as is recollected at this writing, nearly 20 per cent. less than that past the plug when smooth without the grooves.

The design illustrated in Figs. 1 and 2 is entirely new to the author, and he has no recollection of having seen a similar scheme. It is novel and interesting, although in the author's opinion it would not have been a satisfactory device in practice, for the reason that the wings of the rotating cylinder would tend to exhaust the air from the reservoir *a*, even if the air were not absorbed, which it probably would be, and also that the cylinder is seriously out of balance, the thrust being from the inlet pipe *d* to the outlet *ff*.

The meter shown in Figs. 4 and 5 is illustrative of a type upon which much labor and thousands of dollars have been unsuccessfully expended. To render it certain of operation at all rates of flow it would require the addition of the duplex piston and valve of Worthington or the gravitating weight of Kennedy. Without such auxiliary means the reversal of the valves depends upon the momentum of the piston; wherefore if the friction of the piston is equal to or greater than its momentum, the valves, if made to lap the ports, will close off and stop the flow; or if constructed without lap, will permit the fluid to pass free.

But it is comparatively an easy task to criticise an early work through the "hindsight" of after experience. It has been aptly said that "To rightly propose a problem is a considerable step toward its solution;" and such a proposition to the end in view Mr. Leverich at least came very near to making, if he does not in fact submit it in the opening paragraphs of his present discussion.

ERRATA.—Vol. XXV, p. 447.—15th line from top 3 400 yards should read 400 yards.

A late letter from Mr. Edwards says: "The capacity of the dredging steamer 'Advance' is 500 yards, and she loads herself in Gedney's Channel (one of the entrances to New York Harbor) on an average in 40 minutes. The capacity of the 'Reliance' is 650 yards and she loads herself in 45 minutes."

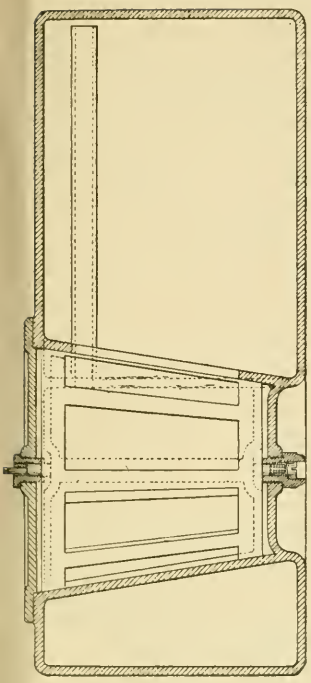

FIG. 6.

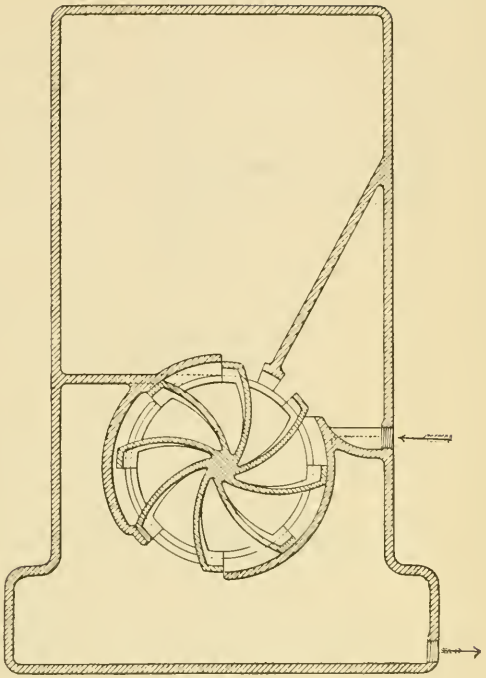

FIG. 7.

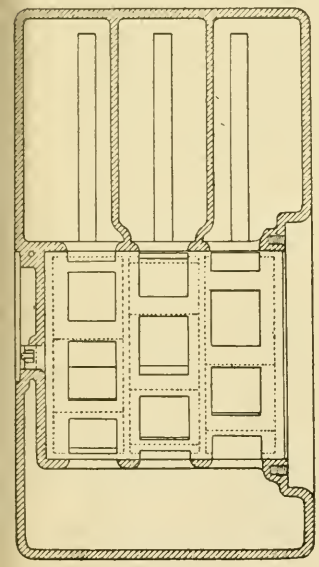

FIG. 8.

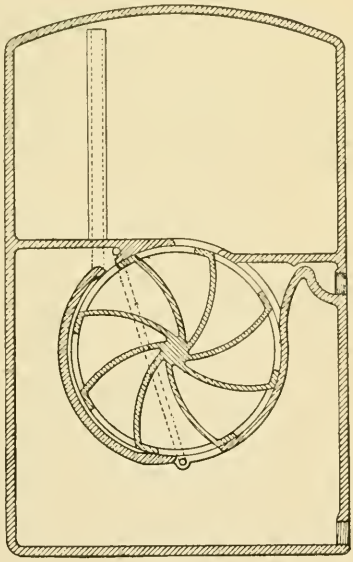

FIG. 9.

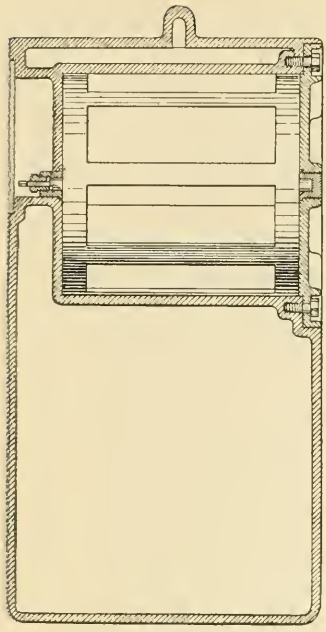

FIG. 10.

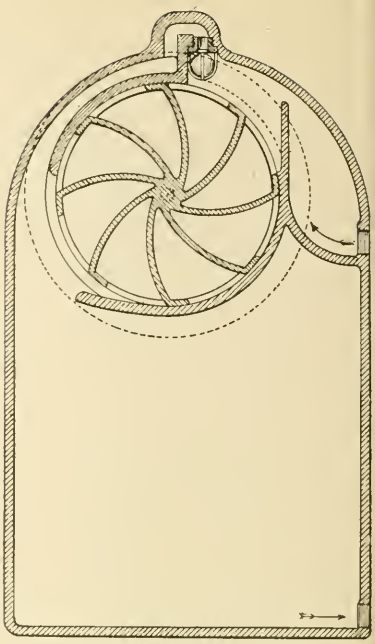

FIG. 11.

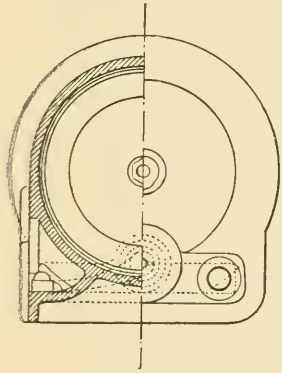

FIG. 12.

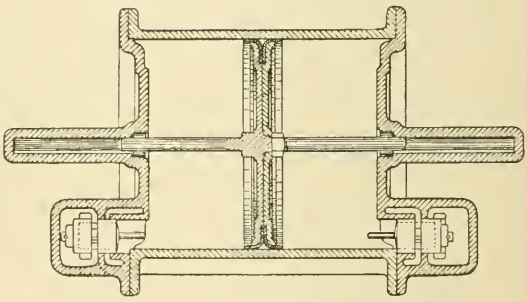

FIG. 13.

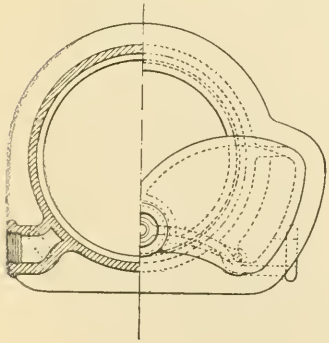

FIG. 14.

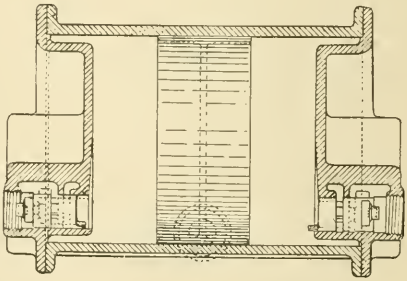

FIG. 15.

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INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

ERRATA.

By an exchange of tracings in preparing the matter for publication, the long scale to the right on plates XCIX and C in the November number, page 537, was erroneously inserted.

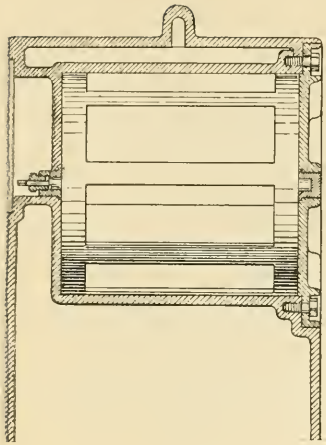
The *correct* scale for each is that given under the title. The other is about 12 per cent. too small.

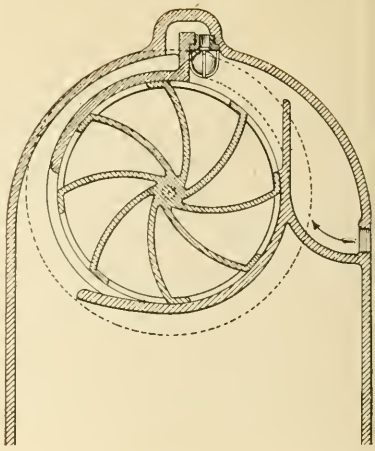
The numbers on plates XCVI and XCVII should be reversed.

all nations is rapidly increasing, it is evident that nothing can be of more importance to the interest, not only of the City of New York, but of the whole country, than an ample waterway between its docks and the ocean, of such depth and width that the largest ships of these and future times may pass into and out of its harbor, at all states of the tide, as safely and freely as ferry-boats are run on its surrounding bays and rivers.

Introductory to this paper it seems desirable to call attention to the commanding importance of the improvement:

"At this port two-thirds of the merchandise imported into the United States are received, and two-thirds of the import duties are collected. From this port are sent out one-half of the domestic products of the country which are exported, and here one-half of the foreign tonnage trading with the United States enters. Three-quarters of the passengers traveling between the United States and foreign countries come and go





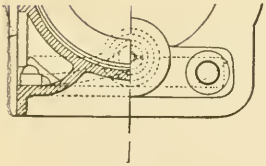


FIG. 12.

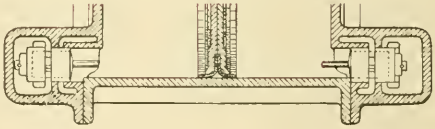


FIG. 13.

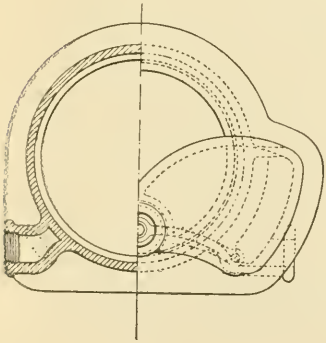
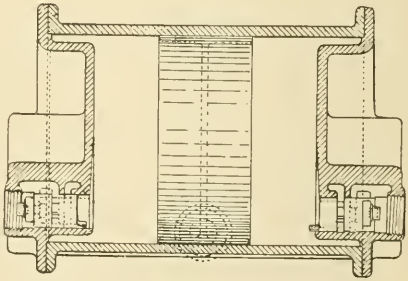


FIG. 14.



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513.
(Vol. XXV.—December, 1891.)

THE IMPROVEMENT OF THE CHANNELS AT THE ENTRANCE TO THE HARBOR OF NEW YORK.

By Joseph Edwards.

When it is considered that the City of New York, with its adjacent cities, is soon to become, if it is not already, the greatest commercial center of the world, and that its social as well as business relations with all nations is rapidly increasing, it is evident that nothing can be of more importance to the interest, not only of the City of New York, but of the whole country, than an ample waterway between its docks and the ocean, of such depth and width that the largest ships of these and future times may pass into and out of its harbor, at all states of the tide, as safely and freely as ferry-boats are run on its surrounding bays and rivers.

Introductory to this paper it seems desirable to call attention to the commanding importance of the improvement:

"At this port two-thirds of the merchandise imported into the United States are received, and two-thirds of the import duties are collected. From this port are sent out one-half of the domestic products of the country which are exported, and here one-half of the foreign tonnage trading with the United States enters. Three-quarters of the passengers traveling between the United States and foreign countries come and go

by way of New York, and three-fifths of all emigration land at Castle Garden."

The following are the statistics of the commerce of the harbor for the fiscal year ending June 30, 1885:

Total receipts from all sources	$126 183 873 57
Value of imports	380 075 748 00
Value of imports for interior ports	16 833 562 00
Value of imports in transit	16 052 183 00
Value of domestic exports	334 729 775 00
Value of foreign exports	9 796 534 00
Value of merchandise in bond June 30th	21 485 083 00
Value of specie (imports)	6 314 264 00
Value of specie (exports)	14 656 718 00

	Tonnage.
Foreign vessels entered, 4 130	4 729 283
Foreign vessels cleared, 4 031	4 670 360
American vessels from foreign ports, 1 691	930 444
American vessels for foreign ports, 1 273	770 105
Coastwise vessels entered, 1 913	1 761 348
Coastwise vessels cleared, 3 076	2 280 063

Channels of New York Harbor.—The several channels of New York Harbor below the Narrows, and the work performed on them, are clearly shown on the accompanying map, which was prepared from a special survey made by Maj. G. L. Gillespie, Corps of U. S. Engineers, in 1884, by direction of the Government, with reference to this improvement, the figures showing the soundings having been changed to correspond with the depth and width of the channels as now improved, and the location of the work designated by checked lines.

Col. Gillespie's report of 1890 states as follows:

"The lower bay is a large tidal basin, with an area of about 100 square miles." The distance by the main ship channel, "from the Battery at New York to 30-foot soundings outside of the bar of Gedney's Channel, is 22 miles, and by the Swash Channel it is 18 miles, or from the Narrows 15 and 11 miles respectively. From the northeast around by the east to the southeast the lower bay is open to the full sweep of the Atlantic Ocean. From the Narrows to the northern point of Sandy Hook is about 9 miles, but the shortest distance across the bay, from the point of the Hook to Coney Island, is 7 miles. Below the Narrows there is one main channel, known as the main ship channel, running

southward to a point about 1 mile west of the upper end of Sandy Hook; thence turning northward and eastward for 4 miles to the head of Gedney's Channel, and thence through Gedney's Channel east to deep water of the ocean. Gedney's Channel is the main channel across the Ocean Bar, lying at the entrance to New York Harbor, about 3 miles outside of Sandy Hook, and east by north from it. The Swash Channel is really a cut-off from the main ship channel, leaving it about 6 miles below the Narrows, and joining it again at the western end of Gedney's Channel."

Lying northward of the above described channels, and extending eastward, are three other lesser channels, the "Coney Island" (which is used only by local excursion boats and small sailing vessels), the "Fourteen-Foot" and the "East Channels," for location and relative position of which see map (Plate CIII).

Relating to the depths of water in the upper bay, the report states:

"From the Narrows northward to New York City there is no water less than 6 fathoms (36 feet) in depth in the main channel."

Condition of the Channels before Improvement.—Colonel G. L. Gillespie, in his report for 1890, says:

"Before the improvement of the Main Ship Channel into New York Harbor was undertaken by the United States, it was obstructed by four shoals as follows:

"*First.*—The outer bar, about 4 000 feet wide, the channel across which is known as Gedney's Channel, where there were depths 23.7 feet in mid-channel and 22.3 feet in the southern half.

"*Second.*—The shoal at the mouth of the Swash Channel, about 4 000 feet wide, where the depth was 24.3 feet.

"The channel across this shoal has been named the Bayside Channel.

"*Third.*—The shoal northwest of Sandy Hook, about 2 000 feet wide, on which the least depth was 26.2 feet.

"*Fourth.*—The shoal in the Main Ship Channel in the lower bay, west of Flynn's Knoll, nearly 3 miles long, on the crest of which the depth was only 23.9 feet in mid-channel, with depth of 22.6 feet within a few hundred feet of the mid-channel range.

"A large proportion of the vast commerce of the port, which is carried on in vessels of great draught, could only cross these channels at, or near, high water."

Referring to the condition of the channels before the improvement was made, Lieutenant-Colonel McFarland, in his report of 1886, says:

"Major Gillespie, in 1884, reported that the deepest draught vessel that had ever entered New York Harbor was the Spanish frigate *Numancia*, which drew 28 feet 8 inches, but that she had to wait for a

spring tide to cross the bar. The minimum depth of water in the Main Ship Channel at mean low water in February, 1886, was 23.3 feet, to which must be added 4.8 feet for mean rise of tide, which would give a minimum depth of 28.1 feet at mean high water, or only one-tenth of a foot more than the draught of the largest ocean steamers at present. Slack water at Sandy Hook averages about forty minutes, so that the vessels of deep draught have only forty minutes in which to cross the bar in Gedney's Channel and to pass the Knolls, 7 miles above it. It is common to see vessels passing over the bar with a wake 500 to 1 000 feet long behind them of material churned up from the bottom by their propeller blades. Thirty feet at mean low water appears to be the depth which is required to enable the largest ocean steamers to come in with 2 feet to spare below their bottom."

First Action Taken by the Government.—Though the need of deeper channels in New York Harbor had been yearly and rapidly increasing, and had been frequently agitated by the commercial bodies of New York, the Government, though from time to time solicited to provide increased facilities, took no action, and up to July 5th, 1884, no money had been appropriated for the improvement of the lower bay and its channels, and no work had been done. But on that date, without any preliminary examination, survey or report, an appropriation of $200,000 was made by Congress for deepening Gedney's Channel, New York Harbor. At this time the engineer in charge, Major G. L. Gillespie, "was authorized and directed to make a survey of the lower harbor from the eastern end of Coney Islannd to Sandy Hook, with a view of determining the most feasible plan of improvement. The survey was begun in August and completed in November, 1884, and transmitted to the Chief of Engineers, December 6th, 1884."

Projects Proposed by Government Engineers.—When the improvement of the harbor thus came to be practically considered by the Government, the problem to be solved by its engineers related to the method to be adopted, which, as far as could be determined without experiment, would best attain the desired results. This involved several considerations bearing upon whatever plan might be projected:

First.—The probability of successfully obtaining the required depth of water.

Second.—The probability of permanency of deeper water after the completion of the work.

Third.—The relative cost of the different plans.

First Project.—Major Gillespie accompanied the report of his survey, above mentioned, with various observations relating to what he found

to be the condition of the channels as compared with their condition at previous times; the velocity of the currents, the character of material found in the different channels, and various other important information relating to the proposed improvement; together with his project for improving the channels and the estimated cost, as will appear from the following extracts from his report for 1885:

"The act of July 5th, 1884, makes a specific appropriation for the deepening of Gedney's Channel, and this project, therefore, is limited to that channel. The methods usually adopted for deepening a channel are by contraction or by removal of material by steam dredges."

Referring to contraction by constructing a stone dike to spring from Coney Island and extend several miles toward Sandy Hook, and strengthening Sandy Hook to prevent it being washed away, Major Gillespie says:

"Before considering such a project it would be well to try the experiment of deepening the channels across the bar by dredging, which has been so successful, by report, at the mouth of the Tyne, and at other ports in England, where the dredged channels have been in the open sea, and when improved, have been self-maintaining. If the experiment fails here, then the composition and nature of all the shoals should be accurately determined by deep borings, the action of the currents definitely learned, and the method of contraction studied.

"The project which I submit for present consideration, then, is to open a channel through the shingle shoal lying across the western entrance to Gedney's Channel, between the 30-foot curves, low stage, by dredging, or by any other method provided for the raising of the obstructing material and carrying it elsewhere to an assigned place of deposit, or by any well developed plan of removal by artificial currents. The proposed cut will extend along the axis of the channel for an approximate distance of 4 000 feet, will be 1 000 feet wide, and will carry 30 feet at mean low stage. The maximum depth of cutting will be 6.5 feet, and the amount of material required to be removed will be 700 000 cubic yards measured in place. I do not know that the cut, once opened, will be self-maintaining, but the present appropriation being small, it is well enough to experiment with it, and if the experiment is moderately successful, the use of large sums for contraction by stone structures may be avoided, and the annual appropriation for maintenance may be placed at comparatively low figures.

"There is a shoal in the main channel west of Flynn's Knoll, 2 miles (approximately) wide, mean low water, with only 25 to 25.5 feet deep. To deepen the channel there to 30 feet mean low water will require an improvement similar to that in Gedney's Channel; but as it is inside of the bar, and there is a safe and deep anchorage for vessels near the Hook,

the improvement is not so urgent as in the latter channel; still, there are other considerations which make it highly important that this shoal should be removed simultaneously with the improvement on the bar."

COST OF THE IMPROVEMENT.

Gedney's Channel:	
Channel 1 000 feet wide, 30 feet mean low water, 700 000 cubic yards, at 50 cents	$350 000
Channel 1 000 feet wide, 28 feet mean low water, 385 000 cubic yards, at 50 cents	192 500
Channel 1 500 feet wide, 27 feet mean low water, 367 000 cubic yards, at 50 cents	183 500
Channel 1 000 feet wide, 27 feet mean low water, 200 000 cubic yards, at 50 cents	100 000
Channel 800 feet wide, 27 feet mean low water, 146 000 cubic yards, at 50 cents	73 000
Main Channel, West of Flynn's Knoll:	
Channel 1 000 feet wide, 30 feet mean low water, 1 550 000 cubic yards, at 40 cents	$620 000
Channel 1 000 feet wide, 28 feet mean low water, 794 000 cubic yards, at 40 cents	317 600
Channel 1 500 feet wide, 27 feet mean low water, 738 000 cubic yards, at 40 cents	295 200
Channel 1 000 feet wide, 27 feet mean low water, 467 000 cubic yards, at 40 cents	186 800
Channel 800 feet wide, 27 feet mean low water, 373 000 cubic yards, at 40 cents	149 200

RECAPITULATION.

Improving Gedney's Channel for 30 feet, mean low water	$350 000
Improving main channel for 30 feet, mean low water	620 000
Total	$970 000

"No allowance has been made in the above computations for irregularities in cutting, or for increase in bulk should the material be measured in scows. This increase would amount probably to 30 per cent. for the 28 and 30-foot depths, and to about 50 per cent. for the 27-foot depths.

"The least width for a conveniently navigable channel, where cross currents exist, for the largest class of vessels, is 800 to 1 000 feet. I

recommend that the available funds be applied toward the opening of the 30-foot channel, by dredging, for a width dependent upon the cost of removal of material, and the work be done by contract, after soliciting sealed proposals by public advertisement in the usual way. I also recommend that Congress be asked to make an additional appropriation of $770 000 this session, to complete the improvement proposed both for Gedney's Channel and for the main channel on the inside, or for application toward the commencement of permanent works of construction, if such works be found necessary. It is my belief that if before any work is done a sufficient time be given to the contractors to prepare an extensive plant, suitable for making the improvement rapidly, the money appropriated will be most judiciously and advantageously expended, and that it may reasonably be expected that the channel will, after the proposed improvement has been effected, be self-sustaining for many years."

It seems reasonable to suppose that the above described project, suggested and recommended by Major Gillespie, determined the method of improving New York Harbor; and though it was at first only experimentally adopted, yet by its final and complete execution it has resulted in preventing an unnecessary outlay of a large sum for contraction works, and avoiding any detriment to the Lower Bay that might have resulted therefrom, as well as consummating the desired results.

Second Project.—Major Gillespie's project and recommendation was submitted, by the Chief of Engineers, to the Board of Engineers, December 10th, 1884. The Board submitted its report thereon December 23d, 1884, which is too extended to be quoted here in full. After concluding that the desired improvement could be best obtained by contraction, it says:

"Dredging would be needed through the 25-foot shoal west of Flynn's Knoll. Colonel Gillespie's estimate for a channel through this shoal, 1 000 feet wide and 30 feet deep, is $620 000. If after the construction of the dike the main interior channels should remain as they now are, this dredging would form a part of the plan; but as that is uncertain, and as much relief would be afforded here by a clear channel 1 000 feet wide and 28 feet deep, in advance of the completion of the 30-foot channel on the bar, dredging to that extent is recommended. Colonel Gillespie estimates its cost at about $420 000. The total cost of the improvement, giving 30 feet deep from New York to the ocean, would be about $5 000 000 or $6 000 000. The Board of Engineers has little expectation that anything more than temporary relief can be obtained by dredging on a bar exposed to the full force of the Atlantic, and hence cannot recommend that method for a permanent improvement. As already stated, it should be used at Flynn's Knoll.

"Having considered the problem of improving the entrance to New York Harbor in a general way, attention will now be turned to Colonel Gillespie's project for expending the $200 000 appropriated by act of Congress approved July 5th, 1884, for Gedney's Channel. While not expecting large results from dredging here, yet, as the appropriation is specifically confined to this channel, the Board recommends that an attempt be made to secure a channel here 28 feet deep, and of such width as the appropriation will pay for, by one of the numerous methods of dredging. If before the whole appropriation were expended experience should show that the dredged channel rapidly filled up, the work might be stopped by the Secretary of War, the contractor being properly remunerated.

"To recapitulate: The Board recommends as a general plan for improving the entrance to New York Harbor so as to give 30 feet from New York to the ocean, the construction of a stone dike running about south south-east from Coney Island to such distance as shall be found necessary, and probably not less than 4 miles; the protection of the head of Sandy Hook; and the dredging of a 30-foot channel from the deep water near Sandy Hook to deep water below the Narrows; also the immediate dredging of a channel 1 000 feet wide and 28 feet deep through the shoal west of Flynn's Knoll, as soon as Congress shall furnish the funds; also that the existing appropriation be applied to dredging Gedney's Channel to a depth of 28 feet."

Increased Estimate.—Referring to Major Gillespie's estimates, made and transmitted to the Chief of Engineers December 6th, 1884, amounting to $970 000, as the probable cost of dredging the two channels, Colonel McFarland, in his report for 1886, states that Major Gillespie's estimates were too small, and says "no allowance is made in them for irregularities of cutting or for increase of bulk, should the material be measured in scows, and that this increase would amount to 30 per cent. for a 30-foot channel. Actual experience in dredging from February, 1886, to June 24th, 1886, shows this increase to be roughly as 55 to 40" (equal to 37½ per cent. more), "so that to Major Gillespie's estimate should be added, in round numbers, $150 000 for Gedney's Channel, and $250 000 for the Knolls, making the totals:

Gedney's Channel	$500 000
The main channel	870 000
Total	$1 370 000

"This is the amount required at Major Gillespie's estimate of fifty cents per cubic yard in Gedney's Channel and forty cents per cubic yard in the main channel."

Why Dredging only was Finally Adopted.—As the appropriation of the $200 000 granted in 1884 was made for dredging in Gedney's Channel, this money could not be applied to the general improvement in any other way; therefore, dredging in this channel was commenced before the project for contraction was either adopted or abandoned. And before the question of contraction, at the estimated cost of $4 500 000, was wholly dismissed, the work that in the mean time had been done and was being done on this channel foreshadowed the possibility of accomplishing the entire contemplated improvement by means of dredging alone. The Board of Engineers, therefore, recommended that the dredging be further extended, and that if the results continued to be favorable and altogether successful, the delay and cost of constructing contraction works might be avoided.

At the beginning of November, 1886, 303 869 cubic yards had been dredged from this channel, and some doubt existing as to whether the shifting of sand during winter storms would not again fill up the cut, arrangements were made for determining this question by having comparative surveys made. One of these was taken between the 17th of October and 2d of November, 1886, and the other in May and June of 1887, by Lieutenant Derby, of the Corps of Engineers, who had been placed in local charge of the work.

The permanency of the results of dredging, as shown by these comparative surveys, will be seen by the following quotations relating thereto, from Colonel McFarland's report of 1887:

"The depths were found to agree almost exactly with the depths given by the survey made in the fall, the only difference being that those obtained in the spring were found to be one or two-tenths of a foot greater. This is a very satisfactory result, for it shows conclusively that for eight months, including the stormiest season of the year, the channel has maintained the increased depth which it had received; and it leads to the belief that the still greater depth which the act of Congress calls for may be equally maintained when once secured."

Besides the above and other evidence that Gedney's Channel could be successfully and permanently improved by dredging alone, as first projected by Colonel Gillespie, the plan by contraction had met with opposition from the New York Chamber of Commerce. There was apprehension that the contemplated contraction works, in the form of a half tide dike springing from Coney Island and extending several miles toward Sandy Hook, would obstruct navigation through other channels

largely employed by coastwise vessels, and that it might otherwise result in damaging the harbor. The communications between the Chamber of Commerce and the Board of Engineers are too lengthy to be here further alluded to. The report of the Board of Engineers, made December 11th, 1886, though somewhat conservative on the project embracing contraction, was as follows:

"The project presented by the Board of Engineers in its report for December 23d, 1884, for securing a permanent low water channel 30 feet deep from the deep water of New York Harbor to the deep water of the Atlantic, by way of Sandy Hook, is the most certain method of accomplishing the end desired; and, as it provided for the expenditure of more than $1 000 000 in dredging, it is recommended that the existing appropriation of $750 000, which was made under the project, and in accordance with the recommendation of the Chief of Engineers, transmitted to Congress by the Secretary of War, be applied wholly to dredging in Gedney's and the main ship channels. It should be stated that the dredging was begun in Gedney's Channel because Congress restricted the use of the appropriation of $200 000 first made to that entrance. Its deepening by dredging having thus been begun by the direction of Congress, should now be thoroughly tried. Should experience show, and only experience can show, that the cost of maintaining a dredged channel through the outer bar will not be excessive in comparison with that of contraction works, then dredging will be by far the best means of permanently improving the entrance, and it would give the speediest relief to navigation. But if the cost of maintaining the channel prove excessive, then the remainder of the plan proposed by the Board of Engineers in 1884 will come up for execution."

Progress and Testing of the Work.—As the work progressed from year to year, various careful comparative surveys were made to ascertain whether the depths of water attained were being maintained, and if not, to what extent they had been diminished. These surveys invariably proved, that while the depth of water had in no part of the channels become less, it had in some places become a trifle greater. The report of the Engineers for 1888 says:

"Surveys made in December, 1887, and May, 1888, show that no shoaling whatever had taken place on the bar in the interval of six months during which no dredging was done there. As a like comparison was made a year ago, with precisely the same result, there are good grounds for expecting that the dredged channel across the bar may maintain its new dimensions by the action of the current alone."

The report for 1889 says:

"A survey of the Main Ship Channel from below the Narrows out

along the improved channel to deep water beyond the bar, was made in June, 1889, and the resulting charts will soon be published for the information of mariners. These charts, which were first made in December, 1888, in separate sheets, covering the several sections of the improved channel, have been eagerly sought after by all the steamship companies of the port, to whom they have been liberally distributed free of charge. The survey just completed shows that the improvement is in the most gratifying condition. There is no indication that Gedney's Channel has shoaled since the last survey of December 29th, 1888, when the least depth in the channel width of 500 feet was 30 feet at mean low water. The Bayside Channel is entirely free from the small shoal spots which formerly existed in it, at or near the eastern entrance to the Swash Channel, and the line of deep water is now direct from the western entrance to Gedney's Channel, westward to the southern entrance to Main Ship Channel opposite to red Buoy No. 10, and the least depth throughout the entire width of 1 000 feet is 30 feet at mean low water.

"The Main Ship Channel, west of Flynn's Knoll, from buoy No. 10, the northern limit of the 30-foot curve in Sandy Hook Bay, to Buoy No. 12, the extreme northern limit of the present improvement, has 29 feet at mean low water between parallel lines 50 feet and 500 feet, respectively, west of the line of buoys *C*2 and *C*6. The 30-foot channel between the same extreme north and south points has an average width of 350 feet. When it is remembered that before this improvement began, in 1885, the least depth in Gedney's Channel was 22.3 feet, in Bayside Channel 24.3 feet, and in the Main Ship Channel, west of Flynn's Knoll, 22.6 feet—all at mean low water—the great results attained by the work just reported will be quite apparent. The noticeable result is that there is now (1889) a navigable channel from the wharves at New York City to the sea, affording 30 feet depth, approximately, at mean low water, and 34.8 feet at high water, and that it is practicable for the largest steamer which visits the port to pass in or out over the bar in fair weather without regard to the tides."

Relating to the durability of the improvement, Colonel Gillespie, still later on, in his report of 1890, says :

"Surveys of all the channels undergoing improvement were made in July, 1889, and again during January and February, 1890. These surveys show that the improvement is in a very satisfactory condition. Gedney's Channel and Bayside Channel (east and west) are practically completed, having a depth of 30 feet at mean low water, for the full projected width of 1 000 feet. The Main Ship Channel west of Flynn's Knoll has a depth of 30 feet, mean low water, for a width of 500 to 800 feet, and a depth of 28 feet for a width of 800 feet throughout. The severe storm of September 9th, 1890, which caused a suspension of work

for one week, does not appear to have had any effect on the improved channels. There is no evidence of shoaling, and the soundings of the various surveys agree so well with one another that it seems highly probable that the improved depths will be well maintained."

At the date of writing this paper there is a continuous channel 30 feet deep at low tide, and 1 000 feet wide from the Narrows to the ocean, and the largest steamships can enter and leave the port at any hour of the day and all states of the tide.

The several appropriations by Government for improvement were as follows :

For Gedney's Channel : by act of July 5th, 1884.....	$200 000
For New York Harbor : by act of August 5th, 1886....	750 000
For New York Harbor : by act of August 11th, 1888...	380 000
For New York Harbor : by act of September 19th, 1890	160 000
Total.....	$1 490 000

Cost of the Improvement.

Estimate for Contraction Jetties.—The project, by means of contraction works, for the improvement of the outer bar, would have involved the construction of a stone dike extending some 4 miles across the shoals of the bay from Coney Island toward Sandy Hook, with a suitable stone protection for the head of Sandy Hook to prevent its being scoured away by the increased current, at an estimated cost of $4 500 000. Besides the requirement of this dike, it was estimated that dredging through the shoals of the Main Ship Channel, which it was supposed the dike would not improve, would have made the cost of the entire improvement, approximately, $6 000 000. (See table.)

Estimate of Cost by Dredging.—As estimated by the Government Engineers, Colonels McFarland and Gillespie, the whole amount of dredging required to complete a continuous channel from the deep water of the ocean to the Narrows, not less than 1 000 feet wide and not less than 30 feet deep at mean low water, would be 4 300 000 cubic yards ; which, at the price (fifty-four cents per cubic yard) paid on the first contract given out on this improvement, would amount to $2 322 000. Respecting the cost of such a channel by dredging alone, Colonel Gillespie, in his report for 1885, estimated fifty cents per cubic yard for Gedney's Channel work and forty cents per cubic yard for that of the Main Ship

Channel. But in 1886 the engineers estimated the cost between thirty-four and thirty-five cents per cubic yard, and that, at this price, the improvement would cost $1 490 000.

Colonel Gillespie, in his report for 1890, says :

"The estimated cost for opening the channel by dredging, revised in 1886, was fixed at $1 370 000, which was again increased in 1887 to $1 490 000."

After the improvement began it was found essential to extend the Main Ship Channel north of Buoy No. 12, to remove the Northwest Shoal, to deepen the Bayside Channel and to extend the Gedney Channel, none of which were included in the original project, and which account for the subsequently increased estimates.

Actual Cost.—The improvement having been made under several different contracts, varying in number of yards and in price per yard, the entire cost is, of course, made up by the aggregate amount paid on all the several contracts, which is $1 285 682 94 for the removal of 4 875 079 cubic yards (instead of 4 300 000, as first estimated it would be necessary to remove), being an average of 26.4 cents per cubic yard, and only 23.37 per cent. of the originally estimated cost.

The tabulated cost is as follows :

	No. of cubic yards removed.	Average price per cubic yard.	Total cost.
Main Ship Channel	3 201 411	25.26 cents.	$808 850 71
Gedney's Channel	1 673 668	28.5 cents.	477 012 23
Totals	4 875 079	26.4 cents.	$1 285 862 94

Economy of Dredging.—By executing the entire work on both channels exclusively by dredging, instead of contraction supplemented by dredging, not only has the Government saved an unnecessary expenditure of $4 594 701 46, as shown by the following tabular statement, but much less time has been required in satisfactorily completing the improvement, and thus sooner providing the much needed better facilities of navigation, as well as avoiding the apprehended detriment to any of the several channels of the harbor.

Besides this large amount of saving by means of dredging, the Government, by a fortuitous circumstance, obtained some advantage by

dredging in portions of the Main Ship Channel from cross currents, as explained by Colonel McFarland in his report for 1888.

By the same tabular statement will be seen the difference between the various estimates of the cost by dredging only, and the actual cost.

The first official estimate (Report for 1885) for dredging Gedney's Channel was fixed at fifty cents per cubic yard, and for the Main Ship Channel at forty cents; while the average actual cost on the Gedney's Channel has been but 28.5 cents per cubic yard, and on the Main Ship Channel 25.26 cents, which aggregates a difference of $831 535 46.

This table also shows that, even at the revised estimate, subsequently fixed at 34.5 cents per cubic yard, including both channels, the improvement has been completed at a cost of $396 039 31 less than the lowest official estimate.

TABULATED STATEMENT, showing the difference between the estimated cost by the several methods proposed and the actual cost by dredging, based on the number of yards (4 875 079) actually removed.

Project Recommended and Location of Work.	Estimated Prices for Dredging.	Estimated Cost.	Actual Cost—all being done by dredging.	Difference—Saving to the Government.
Contraction work for improving Gedney's Channel	.....	$4 500 000 00	$477 012 23	$4 022 987 77
Dredging in Main Ship Channel, 3 201 411 yards	At 40 cents..	1 280 564 40	808 850 71	471 713 69
Totals	.....	$5 780 564 40	$1 285 862 94	$4 494 701 46
Dredging in Gedney's Channel, 1 673 668 yards	At 50 cents..	$836 834 00	$477 012 23	$359 821 77
Dredging in Main Ship Channel, 3 201 411 yards	At 40 cents..	$1 280 564 40	808 850 71	471 713 69
Totals	.....	$2 117 398 40	$1 285 862 94	$831 535 46
Dredging in both channels, 4 875-079 yards	At 34.5 cents	$1 681 902 25	$1 285 862 94	$396 039 31

Referring to the material in Gedney's Channel, Colonel Gillespie in his report for 1885 says:

"The borings just made by a diver show that the obstructing shoal is composed of gravel, coarse gray sand and shells for a depth of 2 feet or more, well compacted, underneath which lies coarse sand, the larger shingle of the size of a pigeon egg being on the crest of the bar, and the underlying sand similar to that of the adjacent beach and shoals."

As to the material in the Main Ship Channel, the engineer in his report for 1888 says:

"This latter material is much more difficult to dredge, not only on account of the large percentage of mud too fine to be caught in the bins, but also on account of its lying so compactly on the bottom, and being consequently much more difficult to raise with the pumps."

Difference Between the Amount of Material Removed and Amount Paid For.—Much of the material used in the Main Ship Channel, consisting of fine sand, clay and sedimentary mud, was so nearly of the same specific gravity as water, that when it became agitated and minutely incorporated therewith, by the action of the pumps and currents in the suction pipes and bins of the ships and scows, it settled so slowly in the receiving bins that a portion of it went overboard with the overflow; and, owing to its light weight, was carried by the cross-currents beyond the walls of the channel, greatly to the benefit of the Government and corresponding disadvantage to the contractor. Referring to these transverse currents and their good effect on the desired results of the work performed, Colonel McFarland in his report for 1888 says:

"The work done during the winter on the shoal in the Main Ship Channel was surveyed April 16th, and 177 935 cubic yards measured in place were found to have been removed from the shoals. The quantity removed by the dredges amounted, however, to only 128 453 cubic yards measured in scows, which would not correspond ordinarily to more than 102 762 cubic yards in place. It is apparent, therefore, that the work of the dredges has been materially supplemented by the currents, in fact to the extent of about 73 per cent. The surface currents of the bay at this point run transverse to the channel instead of along its axis, and the tendency is therefore to carry overflow material upon the adjacent shoals. A survey made in June to ascertain whether this material found a lodgment in the channel at some point further down stream, indicates, on the contrary, that the channel has slightly deepened from natural causes alone, both in the prolongation of the dredged area where the work has been done, and in the dredged area itself where work has been suspended for six weeks. These changes are highly satisfactory as far as they go, both as regards the prospects of permanency in the dredged channel and as regards the great saving that will result in the carrying out the project, if through the assistance of the currents the place measurement continues to exceed the scow measurement."

Thus it is seen that on this part of the work, up to the time of this survey, the contractors improved the channel by removing $1\frac{73}{100}$ cubic yards of material for each cubic yard they were paid for handling.

Contracts.—Contracts were awarded at various times to Roy Stone, the Brainards, Joseph Cummings (of the firm of Morris & Cummings), and the Joseph Edwards Dredging Company. The total number of cubic yards excavated from the channels, taken to sea and deposited outside of the Scotland Light-ship, in not less than 14 fathoms of water, was 4 875 079, at a total cost of $1 285 862 94 and an average cost per yard of $26\frac{4}{10}$ cents. The amount removed by the dredges described in this paper was 4 299 858 yards, at an average of $24\frac{48}{100}$ cents per yard.

It will also be seen by these tables that the above named company removed this large amount of material at an average price of 24.48 cents per cubic yard; and that the general average price paid to other contractors was 40.53 cents per cubic yard, being at the rate of 65.6 per cent. more than was paid this company.

DETAILS OF CONTRACTS WITH JOSEPH EDWARDS DREDGING COMPANY.

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price—cents per cubic yard
GEDNEY'S CHANNEL.						
April 27, 1887...	700 000	Dec. 1, '88.	Dec. 31, '88.	Dec. 22, '88.	770 410	28.5 cents.
Dec. 15, 1888....	600 000	Jan. 1, '90.	.....	Nov. 30, '89.	599 362	17. "
Total.....					1 369 772 yards.	
Average.....					22.84 cents.	
MAIN SHIP CHANNEL.						
May 19, 1887....	1 500 000	Dec. 1, '88.	June 30, '89, and to Dec. 31, '89.	Dec. 3, '89.	1 305 202	28.5 cents.
Mar. 22, 1890...	On Brainard's 1 000 000 yard contract.	June 18, '90, withdrew from the work.	.....	.....	169 754	$16\frac{2}{3}$ "
Mar. 18, 1890....	425 000	Jan. 1, '91.	.....	Aug. 13, '90.	425 000	23.5 "
Aug. 13, 1890....	530 000	June 1, '91.	.....	Feb. 6, '91.	530 000	22.6 "
Feb. 16, 1890....	500 000	Oct. 1, '91.	Nov. 1, '89.	Oct. 10, '91.	500 130	23.9 "
Total.....					2 930 086 yards.	
Average.....					25.25 cents.	

Total Number of Yards and General Average Price.

Gedney's Channel	1 369 772	cubic yards.
Main Ship Channel	2 930 086	" "
Total	4 299 858	cubic yards.

General average price per cubic yard, 24.48 cents.

Details of Contracts with Other Contractors.

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.
Gedney's Channel—(Roy Stone).						
Feb. 7, 1885...	A channel 200 feet wide, 28 feet deep.	June 1, '85.	.....	May 14, '85, Mr. Stone was released from contract.	None.	Taken at 33 cents.
Gedney's Channel—(Elijah Brainard).						
July 31, 1885...	320 000	June 30, '86.	Nov. 1, '86, and to Dec. 31, '86.	Nov., 1886.	303 896	54 cents.
Main Ship Channel—(Brainard Brothers).						
May 11, 1888...	200 000	Dec. 1, '88.	June 30, '89, and to Dec. 31, '89.	Sept. 30, '89.	200 000	28.5 cents.
Main Ship Channel—(Joseph Cummings).						
May 11, 1888...	800 000	Dec. 1, '88.	June 30, '89 —quantity limited to 200 000 cubic yards.	Withdrew from the work.	None.	Taken at 28.5 cents.

MAIN SHIP CHANNEL—(Brainard Dredging Co.).

Nov. 26, 1889...	1 000 000	Withdrew from the work April 16, 1890—Contract annulled June 18, 1890.....	.....	71 325	16⅞ cents.

Total..... 575 221 yards.
Average..... 40.53 cents.

TOTAL NUMBER OF YARDS AND GENERAL AVERAGE PRICE.

Main Ship Channel..... 271 325 cubic yards.
Gedney's Channel..... 303 896 " "

Total number of yards removed..... 575 221 cubic yards.
General average price per cubic yard, 40.53 cents.

Work was begun on the first contract September 26th, 1885, and the work on the last contract was completed October 10th, 1891, covering a period of six years and fourteen days.

History of Contracts.—An account of the attempts, failures and relative success of the several contractors will be seen by the following extracts from the annual reports of Colonel McFarland for 1886 and 1887.

"The exposed position and frequent storms on the bar, the great depth of water, and other unfavorable conditions, made it a difficult matter to decide upon the best mode of deepening Gedney's Channel. It was not expected that ordinary clam-shell and dipper dredges could be used there to advantage, and therefore in drawing up the specifications for the work provisos were inserted requiring bidders to furnish plans and descriptions of the plant they proposed to use, and also a proviso that, if after the plant had been in use a reasonable time, and had not obtained good results, the contracts should be annulled, and the contractor should be reimbursed a fair amount for his outlay.

"Bids for this work were opened January 15th, 1885. The lowest bidder was the firm of Morris & Cummings, who proposed to do the work at thirty-five cents per cubic yard, with clam-shell dredges and centrifugal pumps. But apart from the fact that the bid of Morris & Cummings was irregular, there was a bid from Roy Stone, of New York, for deepening the channel by means of what he termed 'hydraulic plowing,' a process which consists of stirring up the material composing the bottom by means of a strong jet of water thrown against the sandy bottom during the ebb tide, which the projector thought was strong enough in Gedney's Channel to carry away the material so loosened.

"As Mr. Stone's offer carried with it no obligation of payment on the part of the Government, unless he should deepen the channel to 28 feet for a width of 200 feet to begin with, and as it was certain that the ordinary means of dredging could not be successfully used in a place so

much exposed to sea action as Gedney's Channel is, it was decided to accept Mr. Stone's bid; but as very little confidence was felt in the success of the process, though it seemed expedient to give it a trial, the time for completing this contract was limited to June 1st, 1885, and, if at that time satisfactory results had not been obtained, the contract was to be annulled without any payment being made to the contractor. A contract to this effect was therefore entered into with Mr. Stone, February 7th, 1885, and on the 24th of March work under it was begun. From the outset the method was unsuccessful, and after a short time, an induction pipe was substituted for the water jets, which was expected to draw the material up to the surface of the water, and thereby give it a better chance of being carried off by the current. This, however, proved no more successful than the first method, and on the 14th of May, 1885, Mr. Stone was released from the contract at his own request. No money was paid him by the Government.

"Major Gillespie immediately wrote to Morris & Cummings to ask if they wished to enter into a contract under their bid of January, but they refused, and the work was advertised May 21st, 1885, and proposals were again opened, June 23d, 1885. The prices of the bids varied from thirty-three cents to $1 50 per cubic yard. Mr. Stone was again the lowest bidder, at thirty-three cents, but from being unable to prove his ability to execute the contract, his bid was thrown out. The second lowest bid was that of the Atlantic Dredging Company, at 47 cents per cubic yard. Their method was not specific, the date of commencement too remote, and the time for the completion of the contract not given. The same may be said of the third lowest bid, that of Michael Murray, at fifty-three cents per cubic yard, except that his individual responsibility did not seem as good as that of the Atlantic Dredging Company. These two bids were therefore thrown out, and the contract was awarded, with the approval of the Chief of Engineers, to the next lowest bidder, Elijah Brainard, at fifty-four cents per cubic yard.

Mr. Brainard proposed to put upon the work, within two weeks of signing the contract, three clam shell dredges, with a capacity of 4 000 cubic yards per day, and further to build and put on within three months of the date of opening the bids a hydraulic pump excavator, with an estimated capacity of 2 500 yards per day. The contract with Mr. Brainard was signed July 31st, 1885, but as it was found impracticable to keep a scow alongside of the clam-shell dredges in the rough water on the bar, it was not thought advisable to attempt to put them to work. On the 26th of September, 1885, the hydraulic excavator was ready and began work. It was expected by the contractor that this machine would raise from 10 to 20 per cent. of solid material, which would correspond to 2 000 to 3 000 cubic yards per day. The amount actually raised was less than 5 per cent.

Beginning September 26th, 1885, only fifty-seven hours and forty minutes actual pumping was done up to December 23d, 1885, when, in a storm, going out to the dumping ground, the A-frames, derrick-frames and suction pipe were carried away, and the excavator had to be laid up for repairs. In the fifty-seven hours forty minutes pumping, 6 411 cubic yards of solid material were excavated. The repairs consisted of replacing the old engine with a new one of double power, the derrick-frames were rebuilt and strengthened, the suction pipe was reinforced with a truss, condensers were put in, and various minor repairs and improvements were made, and the excavator was ready for work again February 24th, 1886. Meanwhile, under date of January 2d, 1886, Major Gillespie notified the contractor by letter that the plant did not come up to the capacity called for by the specifications."

Mr. Brainard replied that it was his intention soon to work additional plant of much greater capacity, and that with the experience already had, he anticipated excavating with the new plant from 3 000 to 3 500 cubic yards per diem.

"The excavator was not finally finished until May 29th, 1886. The hull was especially built for the purpose; its dimensions are 152 feet by 44 feet, and it draws about 16 feet loaded; it is scow built, with six pockets and bottom dump valves. The capacity of the pump, as estimated by its makers, is 36 000 gallons per minute, through a 22-inch suction and a 22-inch discharge pipe. Her loaded capacity is 1 055 cubic yards when full. She is not self-propelling, but is towed.

"The *Howard*, another barge which was working last season on Charleston Bar, is self-propelling, and is about 110 feet by 32 feet, and draws about 10 feet. An Edwards pump, capacity unknown, is set low amidships. There are two suctions, one on each side, 9 inches in diameter. The load is carried on an upper deck forward, in bins, which dump through side doors. Her full loaded capacity is about 175 cubic yards.

"The following is a statement of the work of the three excavators since they commenced work under this contract:

	Cubic Yards.
Excavator No. 1	63 935.0
Excavator No. 2	44 684.7
Excavator *Howard*	29 581.4
Total for about 1 year's work to July 31, 1886	138 201.1

"The contract had to be extended, and the work was finally completed and the contract was closed early in November, 1886. The amount of material removed was 303 869 cubic yards."

When it was concluded by the Government that the entire improvement was to be made by dredging, it was found that contractors hesitated

to enter upon the undertaking, as will best be seen by the following account taken from Colonel McFarland's report for 1887:

"In accordance with the recommendation of the Board of Engineers approved by the Secretary of War December 27th, 1886, under date of January 22d, 1887, proposals for deepening Gedney's Channel were called for by public advertisement, the bids to be opened February 23d, 1887. Proposals for deepening the Main Ship Channel were to be opened at the same time, under an advertisement dated January 14th, asking for them. The result of the bidding, however, was not satisfactory. Only one bid was received for the work on the Main Ship Channel, and as this proposed the use of experimental plant, which was forbidden by the specification, it was rejected. Five bids were received for the Gedney's Channel work, but as it seems likely, in view of the failure of the contractors to bid upon the Main Ship Channel work, which was much the larger part, that the Government might have to build its own plant and to do the work itself, it was deemed inexpedient to let a part of it only, and the bids for Gedney's Channel were rejected also."

In view of this situation of affairs, the report goes on to say:

"A circular letter was issued by Colonel McFarland and sent not only to every one who had previously bid upon the work, but to every one to whom the original specifications had been sent, and to others besides, asking them to state under what circumstances, at what rates and in what time they would undertake to do the work of deepening either or both the Main Ship Channel and Gedney's Channel. That circular was as follows:"

"GENTLEMEN,—It is evident from the absence of bids for dredging the Main Ship Channel, New York Harbor, asked for under my advertisement of January 22d, 1887, that there must have been some limitation in the published specifications which those who are accustomed to doing that kind of work were unwilling to accept.

"With the view of getting the work done as speedily and cheaply as possible, I should be glad to have those who are engaged in the business of dredging, or who are accustomed to the management of large contracts, communicate with me in writing, stating under what circumstances and at what rates and in what time they would be willing to undertake and to complete the deepening of either or both the Main Ship Channel and Gedney's Channel, New York Harbor, the total amount to be expended on both of these improvements being about $650 000, and the amount of material to be removed being between 2 000 000 and 3 000 000 cubic yards.

"It is possible that some offer may be made which would be acceptable and which would render it unnecessary for the Government to undertake to do the work itself.

"A free expression of views is invited. As it is important to have the work done as soon as possible, it is desirable to have the matter settled by the 10th of March."

"In answer to this circular ten communications were received, some

for doing the whole work, some for the Main Ship Channel alone, and some for Gedney's Channel alone, all varying in rates, time and method of doing the work. None of the offers for the whole work were entirely satisfactory, however; for although some of the rates offered were low, the time for the completion of the work was too great; but for the Gedney's Channel work, for which only six offers were made, a lower bid than any previously offered was received, and by authority of the Secretary of War it was accepted, and an agreement in accordance with it, dated April 21st, 1887, was made with the Joseph Edwards Dredging Company for removing 700 000 cubic yards of material from Gedney's Channel, at twenty-eight and a half cents per cubic yard. This was six and a quarter cents a yard lower than the lowest offer previously received under advertisement of January 22d, 1887." Two offers were also received for the Main Ship Channel work, but as one of these was for a part of it only, and the other involved the use of experimental plant, both were rejected.

"On the 22d of April, the Joseph Edwards Dredging Company made a proposal in writing to do the Main Ship Channel work at the same rate at which they agreed to do the Gedney's Channel work, namely, at twenty-eight and one-half cents per cubic yard, suggesting at the same time that if their offer were accepted, the additional plant designed for working in the Main Ship Channel might, if desired, be united with that intended for Gedney's Channel, and the work upon that channel might then be completed before the close of the present year; and afterward the combined plant working upon the Main Ship Channel could finish the deepening of that channel within the succeeding year, the period fixed for its completion by the specifications previously issued. A copy of this letter accompanies this report.

"This offer was so favorable, so much time had already been lost, and the deepening of Gedney's Channel before another winter should set in was so very desirable in an engineering point of view and so impossible of accomplishment in any other way than that proposed, that the offer was immediately forwarded to Washington, with a strong recommendation that it be accepted; but it was decided by the Secretary of War that this could not be done, and that the work must be advertised again. This was accordingly done under date of May 3, and bids were opened May 18, 1887.

"Only two were received, and the lowest of these was from the Joseph Edwards Dredging Company, the rate being the same as in the previous offer. This offer was accepted, and a contract was entered into with them, dated May 19th, 1887, for removing 1 500 000 cubic yards of material from the Main Ship or the Swash Channel, as might be found most expedient, at twenty-eight and one-half cents per cubic yard.

"The whole work, therefore, of improving Gedney's Channel and the Main Ship Channel is now in the hands of this one firm, the manag-

ing head, Mr. Edwards, having had large experience with similar dredging methods on ocean bars, under General Gilmore, at the mouth of the St. John's River, Florida, in 1871, and subsequently under Captain Eads in his operations at the mouth of the Mississippi River.

"The plant to be used on the two channels will consist of three self-propelling hydraulic dredges, of a combined daily capacity of 10 000 cubic yards or over. The first one, the *Bolivar*" (subsequently named *Advance*), "which is to be ready July 1st, is a steam vessel, furnished with two 15-inch centrifugal pumps, and capable of carrying a load of 450 cubic yards. The second steamer, the *Mount Waldo*, which is to be ready August 1st, is to have two 15-inch centrifugal pumps and a carrying capacity of 350 cubic yards, and is to be accompanied by four scows, each of 500 cubic yards capacity, each towed by its own tug. The third dredge, the *Reliance*, which is to be ready September 1st, is a new steamer, built for the purpose, and is to have two 8-inch centrifugal pumps and a carrying capacity of 650 cubic yards."

As appears by the engineer's report for 1889: "On December 8th, 1888, a proposition was received from the Joseph Edwards Dredging Company to dredge from the Gedney's Channel division about 600 000 cubic yards, which, it was estimated, would complete the improvement projected for that division, at seventeen cents per cubic yard, in accordance with the specifications governing the execution of their contract which had just expired, with the reservation that no deduction should be made for over-depths of less than 1 foot from the payment due for work done under the agreement. This proposition was approved by the Chief of Engineers December 15th, 1888, and an agreement was accordingly made, with the limitation that the work should be completed by January 1st, 1890."

As appears by the Engineer's report for 1890:

"Proposals were invited by public advertisement and opened March 13th, 1890, for the removal of 500 000 cubic yards of material from the Main Ship Channel north of Buoy No. 12. But one bid, presented by the Joseph Edwards Dredging Company, at 23.5 cents per cubic yard, was received, and being considered reasonable, a contract was entered into with this company March 18th, 1890, approved by the Chief of Engineers Aprill 11th, 1890, for the removal of 425 000 cubic yards.

"Operations under this contract began with the dredge *Advance* April 2d, with the dredge *Reliance* April 4th, and with the dredge *Mount Waldo* June 18th, 1890; and the work was finished August 13th, 1890. On the same date the company contracted to remove 530 000 cubic yards from the Main Ship Channel at 22.6 cents per cubic yard. Under this contract the number of yards removed was the same as the amount called for; and the work was completed February 6th, 1891. February 16th, 1891, a contract was awarded the same company for the removal of 500 000 cubic yards from the Main Ship Channel at 23.9 cents per cubic

yard. Under this contract the number of yards of material removed was 500 130, and the work was finished October 10th, 1891; and with the completion of this contract was completed the improvement of New York Harbor, the company having executed about 88 per cent of the work."

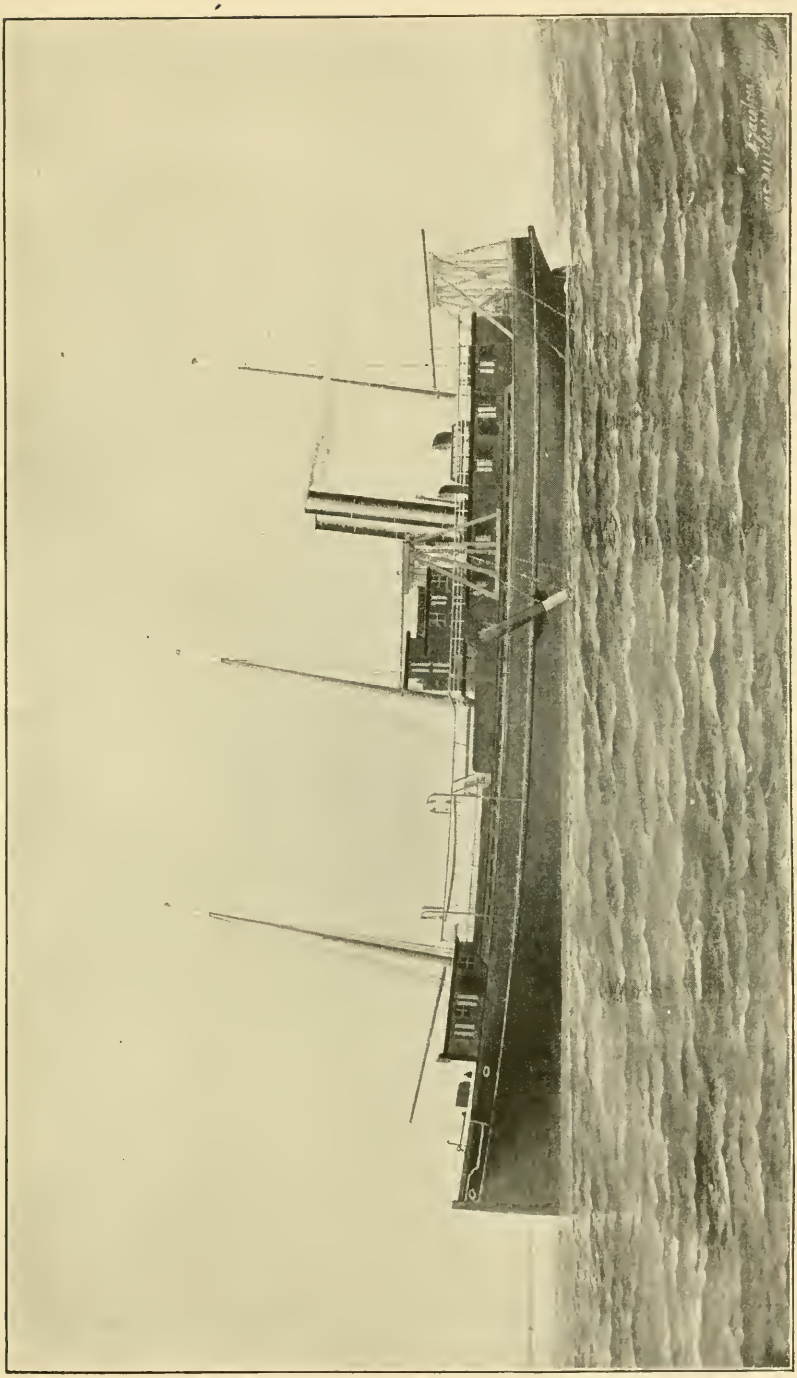
It is not necessary to enter into the details of the other contracts, as but little was accomplished by the contractors.

The magnitude of the undertaking will be seen by considering that besides the excavation of about 5 000 000 cubic yards of material and its transportation out to sea (the average round trip being 21 miles), there had to be dredged, as before explained, a large amount more which went overboard with the overflow, especially in removing the crust of mud and clay in the Main Ship Channel. This material had to be raised from a depth of 24 to 35 feet under water and elevated to a height on ship-board which, from the bed of the channels, amounted to an elevation of from 36 to 46 feet, according to the state of the tide and the depth of the channel. For each cubic yard of solid material thus handled there had to be raised many cubic yards of water, at a height, from the surface of the bay to the mouth of the discharge pipes, of from 10 to 15 feet. The mixture of mud, clay, sand and water thus raised amounted to many times the cubic yards of material which the contractor was paid for handling.

System of Dredging Employed.—The system of dredging employed was what is known as that form of hydraulic dredging in which the means for excavating and elevating the material to be removed are centrifugal pumps, illustrations of those employed being shown in Figs. 1 to 5.

But besides the employment of centrifugal pumps, it was necessary to devise the plant in such a manner as to adapt it to the peculiarities of the work, having reference to the exposed position, depth of water, etc.

The plant provided for executing the work consisted of three sea-going dredging steamers, four large scows, four steam tugs for towing the scows, one steam supply boat, one steam tender, docks, repair shops, storehouses, water-works, coal bins, etc. The dredging steamers, known as the *Reliance* (see Plate CIV), the *Advance* and *Mount Waldo,* are not essentially unlike other sea-going steamers, aside from their dredging outfits. Their dimensions and daily working capacity are as follows:



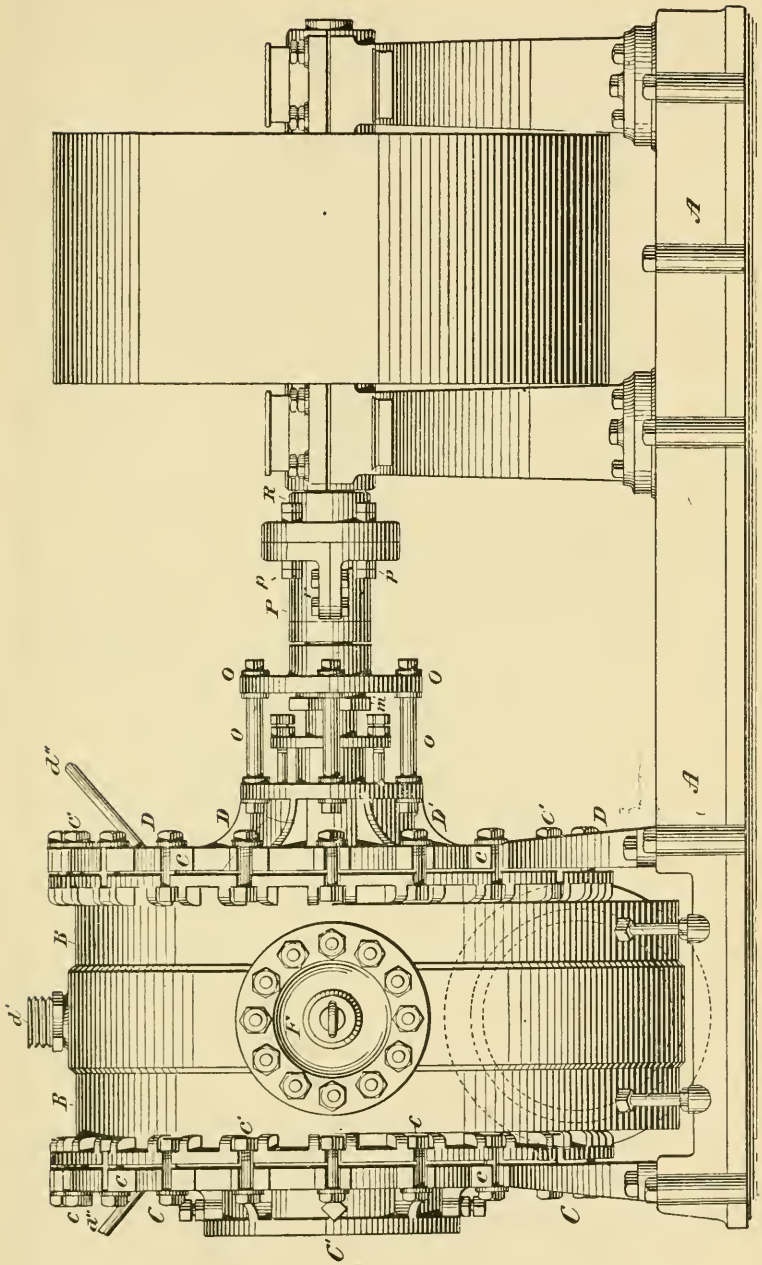


FIG. 1.—Exterior View of Edwards' Cataract Pump.

THE *Reliance.*

Length	157 feet.
Beam	37 "
Depth of hold	16 "
Twin propellers and compound engines.	
Carrying capacity	650 cubic yards.

The daily working capacity of this dredge on Gedney's Channel, where the material consisted of coarse sand, and freighting it about six miles each way, was seven loads; and on the Main Ship Channel, where the material consisted of mud, clay and fine sand, and freighting it a distance of about 12 miles each way, was three loads.

THE *Advance.*

Length	132 feet
Beam	34 "
Depth of hold	8 "
Single propeller and compound engine.	
Carrying capacity	500 cubic yards.

The daily working capacity of this dredge, on the same channels, respectively, was also seven loads and three loads.

THE *Mount Waldo.*

Length	145 feet.
Beam	31 "
Depth of hold	11 "
Single propeller and compound engine.	
Carrying capacity	275 cubic yards.

The daily working capacity of this dredge, discharging into scows, working on the Main Ship Channel, on material consisting of mud, clay and fine sand, was eight scowloads of 500 yards each. The *Mount Waldo*, except in the winter-time, instead of being employed as the other two steamers were, namely, for dredging and transportation, was kept constantly employed at dredging by discharging the material into scows and not into her own bins; the scows were towed to the dumping ground by steam tugs. Besides the three above described steamer dredges and four scows, there were four powerful sea-going tugs for towing the scows to sea; also a large steam lighter employed as a

supply boat for furnishing the dredging steamers with coal, water, duplicate parts, provisions, etc.; and also a steam tug employed as a general tender for convenience of the General Manager and Superintendent.

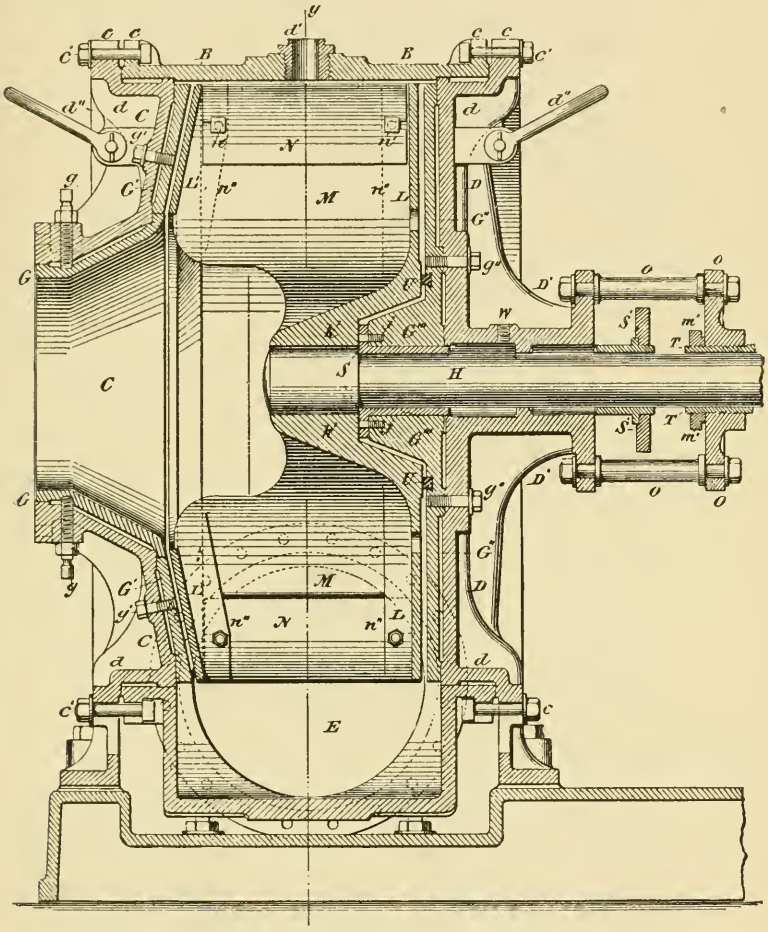


FIG. 2.—Central Vertical Section of Pump.

Each steam dredge was provided with two pumping outfits independently arranged, so that either could be operated regardless of the other; each outfit having a centrifugal pump, engine, suction and discharge pipes. The suction pipes, corresponding to the size of the pumps, one on either side of the steamer, were located about mid-

ships, and extend laterally from the pumps to the outside of the ship; then they turn with an easy bend at right angles, extend (when not in use) along the sides of the dredge, and are held up, lowered and raised by suitable hoisting purchases worked by hoisting engines.

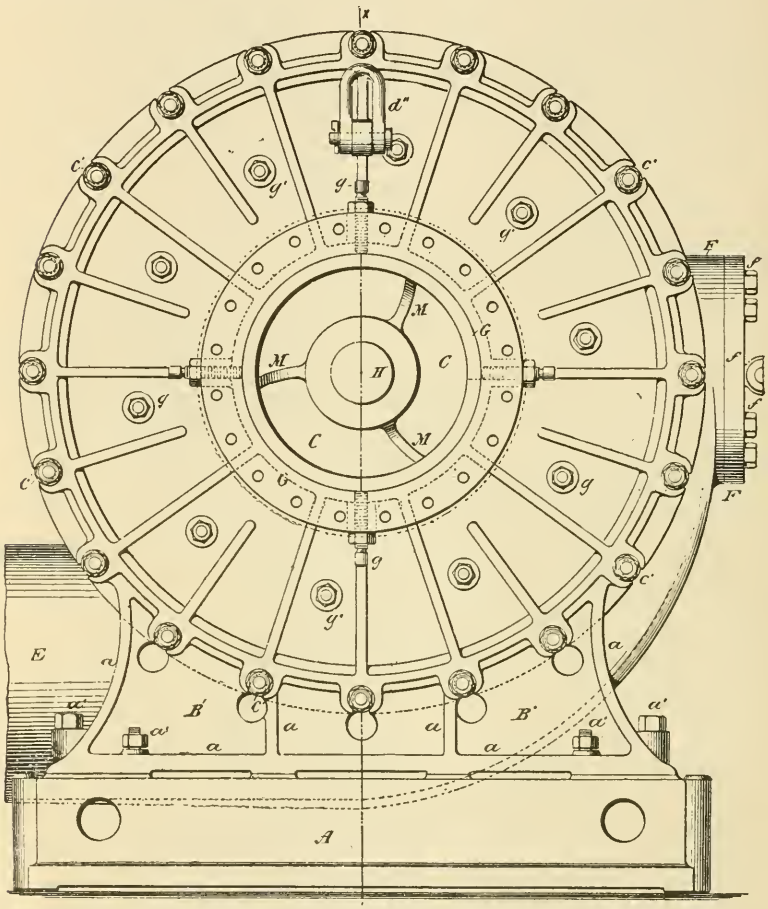


Fig. 3.—View of Suction End of Pumps.

These suction pipes, 15 and 18 inches in diameter, are about 60 feet long, terminating about opposite the stern of the ship, with suitable mouth pieces, termed drags, to fit on the bed of the channel and facilitate the ingress of the material. (See Plates CV, CVI, CVII, CVIII.)

To render the suction pipes flexible, so as to accommodate them-

selves to the pitching and rolling motions of the steamer, a section of them about 12 feet in length, located a few feet from the bend, consists of rubber; these flexible sections being supported by a special hanging arrangement of chains, blocks and hoisting purchases against the vertical strain caused by the weight of the suction pipes themselves and

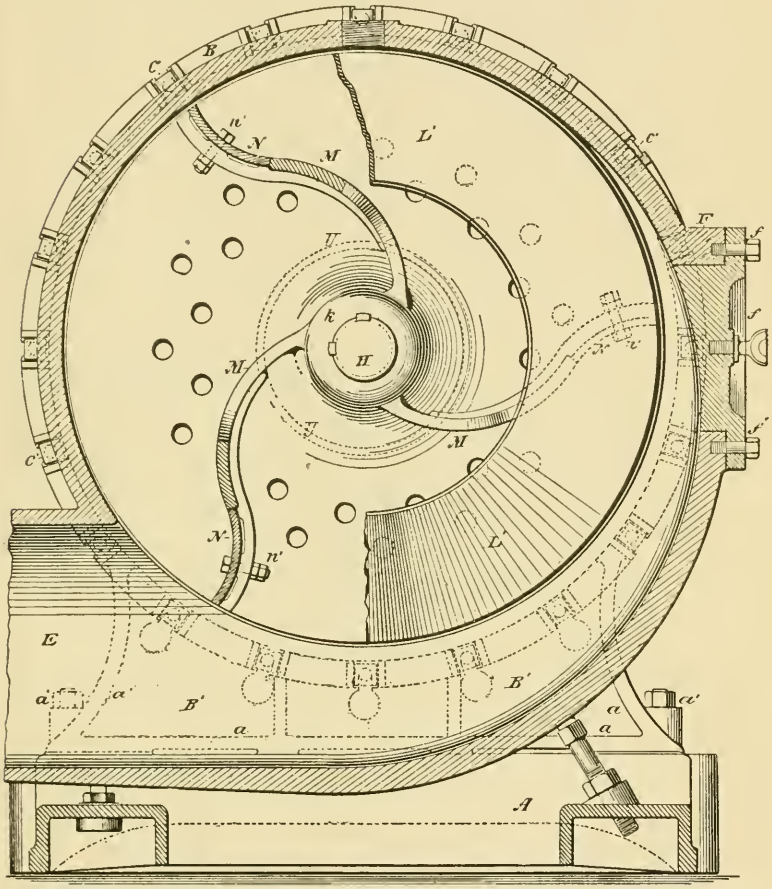


FIG. 4.—Interior View of the Pump, Showing Detachable Plates in Place.

what passes through them; and by tension chains against the longitudinal strain of the drags resting on the bottom. The steamers and scows were provided with special arrangements to afford a long flow of the mixture of mud, clay, sand and water, between the discharge from the pumps and the overflow outlets, to facilitate the settlement of the solid material in their bins.

The scows were divided into compartments surmounted with longitudinal sluice-ways extending either way from a central receiving hopper. These sluices were provided along their course with a series of adjustable bottom and side gates by means of which the material could be deposited faster or slower in the different compartments; whereby the load could be uniformly distributed throughout the length and breadth of the scows, to prevent them from listing and enable them to be freighted to their full capacity. (See Plates CIX, CX.)

The steamers were kept under headway from the time they left their anchorage in the morning until they returned to it at night. When a dredge reached the channel the suction pipes were lowered to an angle of from 30 to 40 degrees, to bring the drags in contact with the bottom. To keep the vessel on her line of work and supply the drags with material, she was kept constantly under steering headway. As soon as the sand bins were filled, the suction pipes were hoisted out of water, and the steamer put under full headway for the dumping ground. While she was turning to return to the work again, the dump gates were opened and her cargo discharged—the discharge being facilitated by pumping water into the bins with the dredging pumps. On again reaching the work her speed was slackened, drags lowered and pumps started—and so on, until the time to return to her anchorage for the night.

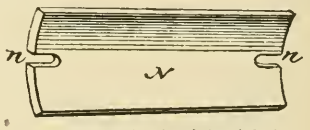


Fig. 5.—Detachable Plate at Outer End of Pump Vanes.

The above brief general description affords but a limited view of the construction and performance of the plant. To fully appreciate the arrangement and effective execution of the steamer dredges, they need to be seen in operation, especially in a heavy seaway. Some idea may be had of the extent of the plant from the fact that in connection therewith in one way and another, it included no less than eighty different steam cylinders.

Difficulties Encountered.—The lower bay, in which the Main Ship Channel is located, embraces an area of about 100 square miles, estimating inside, northward and westward, of a line drawn from Sandy Hook to Coney Island, with an opening outward to the ocean (between Sandy Hook and Coney Island) 7 miles wide. Through this opening, the work on the Main Ship Channel was exposed to the full sweep of the ocean from the northward around by the east to the southeast, and from the

southwest and west to winds sweeping down Raritan Bay, while winds from the northwest and north had scope across and down the channel from Staten Island and the Narrows. Thus exposed to the wind from every quarter, except the south, it was often impossible to handle and tow the scows or even to work the dredging steamers, owing to the roughness of the bay, even in clear weather.

The Gedney's Channel being outside of the entrance to the bay, may be considered as located in the Atlantic itself, and therefore the work on this channel was exposed to the winds and roughness of the ocean from all directions, making it impossible to work here even as constantly as on the Main Ship Channel; while it was impossible to work on either channel during an easterly storm, and sometimes for days after, owing to the continuation of an incoming rolling sea; and often when the dredges did work they were more or less belabored with rough seas.

The dumping ground being outside of the Scotland Light-ship, the material removed from the Main Ship Channel had to be transported an average distance of 12 to 14 miles, making the round trip 26 to 28 miles, and that from the Gedney's Channel 6 to 7 miles, making the round trip 12 to 14 miles. As regards the transportation by towing, there was nearly always sufficient sea on to more or less strain the scows, and require a powerful tug to handle each separate scow.

To avoid liability of collision with passing vessels, especially with large ocean steamers, it was frequently necessary to shift the position and stop the work of the dredges until the steamers had passed. Besides this, these larger steamers produced a disturbing wake of the water, and passing so frequently, interfered not a little with the work. Notwithstanding the utmost caution, two collisions occurred, and one of the company's ships, the *Advance*, was run down, sunk and destroyed during the progress of the work.

On account of rough sea, heavy storms and otherwise inclement weather, as also on account of thick and foggy atmosphere, even though there were no storm or roughness of sea, and sometimes because of running ice, it was often necessary to suspend operations of the entire plant from one to several days with no diminution of expense save a trifle on fuel; such delays hindered the progress of the work and diminished the chances of profit to the company.

As might be expected, in channels so long and extensively navigated, there were frequently found minor yet troublesome obstructions, as

fragments of wrecks, anchors, chains, bars of iron, cannon balls, etc., which when encountered sometimes caused damage to the pumps, suction pipes and drags. Such objects, encountered and passed over, would again and again be found in the way, necessitating a search for them and their removal. Sometimes before being definitely located and removed, they would not only break a drag, but carry away both drag and suction pipe, and delay the operation of the entire dredge for some days, resulting in a loss to the contractor of several thousand dollars.

The constant transverse currents that set nearly at right angles across that portion of the Main Ship Channel extending north and south, necessitated the heading of the dredging ships in a diagonal direction to the line of the channel, thus causing the cross currents to carry one of the suction pipes with its drag or mouth away from the side, and the other under the bottom of the dredging steamers. This was a serious and constant interference with their most successful operation, and frequently caused injury to the suction pipes, sometimes badly breaking them.

The composition of a part of the material in the Main Ship Channel was of such a nature, being nearly of the same specific gravity as water, that, when it became thoroughly agitated and incorporated with the large amount of water handled, only a variable percentage of the amount dredged would settle in the scows and bins of the steamers, while the balance of it would go overboard with the overflow—the respective proportions that would settle and overflow depending, of course, on the nature of the material being dredged.

The extent and nature of the plant, and the character and location of the work performed, rendered it impossible to avoid numerous accidents and break downs, thus necessitating the keeping on hand of a large supply of duplicate parts to diminish delay of repairs, but which greatly increased the cost of the general outfit.

Owing to the tension to which the mechanism of the plant was submitted, and the speed at which it was driven, the cutting effect of sand rapidly forced through the suction pipes and pumps and scattered by wind and spray throughout the machinery, and the strain upon the ships incident to being constantly loaded and unloaded,—the extensive wear and depreciation of the plant constituted a serious drawback to the contractor, it becoming necessary for the outfit to

undergo general yearly repairs amounting sometimes to a cost of fifty thousand dollars.

On pages 606 and 607 is a statement of the work done by the *Reliance* in Gedney's Channel in twenty-eight and one-half working days in September and October 1891, taken from the daily records:

Average time pumping per load.....48$\frac{6}{10}$ minutes.
" cubic yards per load.....584.87 cubic yards.
" time pumping per day.....4 hours 58$\frac{4}{10}$ minutes.
" " on bar per day.....5 hours 43$\frac{4}{10}$ minutes.
" " bar to dump.....34 minutes.
" " dumping.....12$\frac{1}{10}$ minutes.
" " dump to bar.....25$\frac{7}{10}$ minutes.
" " " to anchor.....66$\frac{6}{10}$ minutes.
" " anchor to bar.....51 minutes.
" " under steam per day.....16 hours 4$\frac{2}{10}$ minutes.
" No. of loads per day worked.....6.73
" cubic yards per day worked.....3 936.65 cubic yards.
" rate per minute.....12.03 " "
Time lost by repairs.....2 hours 24 minutes.
" " " weather.....32 " 50 "
Total time lost.....35 " 14 "
in a total of 458 working hours in 28½ working days.

September 10th, 4 790 yards, 8 loads, or at the rate of 16.43 cubic yards per minute. The 6th load of 611 yards was done at the rate of 15.27 yards per minute, and the 7th load of 608 yards was done at the rate of 16.43 yards per minute or an average of 946 cubic feet per hour.

September 11th, 4 582 yards, 7 loads, or at the rate of 15.27 cubic yards per minute.

Government Engineers in Charge.—The engineers under whose direction the improvement of New York Harbor was conducted were Major (subsequently Lieutenant-Colonel) George L. Gillespie and Lieutenant-Colonel Walter McFarland, Corps of Engineers, U. S. Army. Colonel Gillespie being in charge at the commencement, made the preliminary survey, and projected and recommended the plan by which the improvement has been expeditiously, successfully and satisfactorily accomplished, and at a cost to the Government not exceeding 25 per cent. of

Date.	Cubic yards excavated.	Time pumping.	Rate per minute.	Total time on bar.	Time Lost.										Direction and force of wind.	Condition of sea.	No. of loads.	Remarks.
					Turning.	Bar to dump.	Dumping.	Dump to bar.	Dump to anchor.	Off cut and other causes.	Anchor to bar.	By repairs.	By weather.	Total time.				
		H. M.	Cubic yards	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M				
1891.																		
Aug. 27	2 150	3—08	11.44	3—29	0—10	2—15	0—56	1—12	1—05	0—11	.....	.....	.....	8—58	S. E. to S. S. W.	Smooth to Chop	4	One load from Main Ship Channel in the morning.
28	4 174	5—28	12.72	6—04	0—18	3—51	1—36	2—23	1—12	0—18	0—47	.....	.....	16—25	S. W. to N. W.	Chop	7	
29	4 196	5—43	12.23	6—22	0—33	3—44	1—27	2—32	1—17	0—06	0—38	.....	.....	16—00	N. W. to S. W.	Smooth	7	Week ending Aug. 29th, 3½ days on Main Ship Channel.
31	1 666	2—20	11.82	2—37	0—11	1—26	0—39	0—43	1—25	8—04	1—02	.....	.....	15—50	Fresh. S. E.	Swell.	3	
Sept. 1	4 563	5—42	13.34	6—44	0—26	3—54	1—31	2—45	0—55	0—36	1—08	.....	.....	16—57	Light. N. N. E.	Swell.	8	
2	4 344	5—50	12.40	7—03	0—32	3—49	1—19	2—41	0—52	0—41	1—07	.....	.....	16—51	Light. N. W.	Smooth	7	Foggy in the morning.
3	4 376	6—17	11.60	7—16	0—31	3—46	1—20	2—14	0—52	0—28	0—43	.....	.....	16—11	Light. W. S. W.	Smooth	7	
4	4 341	6—21	11.40	7—27	0—30	4—12	1—25	2—03	0—54	0—36	0—42	.....	.....	16—43	Fresh E. S. E.	Rough.	7	
5	1 368	1—41	13.54	2—10	0—14	1—38	1—32	0—42	1—46	0—15	0—54	.....	8—58	17—40	Strong N. E.	Very Rough	3	Too rough to work.
7	4 056	6—04	11.14	6—58	0—18	3—57	1—24	2—34	0—56	0—36	1—06	.....	.....	16—55	Light. N. W.	Chop.	7	
8	4 046	5—35	12.08	6—17	0—21	3—53	1—36	2—28	1—00	0—21	0—32	0—58	.....	16—44	N. W.	Smooth	7	Valve-stem of large washout pump broke.
9	3 148	5—06	10.28	7—32	0—26	2—57	1—23	2—18	0—58	0—34	0—52	1—26	.....	16—00	N. W.	Smooth	6	Belt on port pump broke.

	10	4 790	6—24	12.47	7—13	0—30	3—53	1—37	2—42	1—02	0—19	0—45	.....	.....	17—12	N. W.	Calm.	8	Sixth load (of 611 cubic yards) was at the rate of 15.27 cubic yards per minute.
	11	4 582	6—10	12.38	7—17	0—34	3—55	1—29	2—34	1—02	0—23	0—33	.....	.....	16—40	Light N. W. to S. W.	Calm to Chop.	8	Seventh load (of 608 cubic yards) was at the rate of 16.43 cubic yards per minute.
	12	4 055	6—33	10.32	7—47	0—37	3—30	1—27	2—09	1—27	0—37	0—33	.....	.....	16—53	Brisk. S. E.	Chop.	7	
	14	3 510	6—16	9.33	7—20	0—39	3—25	1—18	2—29	1—19	0—25	1—11	.....	.....	17—02	Brisk. N.	Heavy Chop.	7	
	15	4 076	6—30	10.45	7—43	0—30	3—38	1—24	2—16	1—06	0—43	0—48	.....	.....	16—55	Light. N. W.	Smooth	7	
	16	4 112	6—26	10.65	7—26	0—37	3—25	1—25	2—06	1—12	0—23	0—51	.....	.....	16—25	Light. N. W.	Smooth	7	
	17	4 296	4—44	15.12	5—17	0—16	4—43	1—13	2—59	0—58	0—17	1—00	.....	.....	16—10	Fresh. S. S W.	Chop.	7	
	18	4 232	4—46	14.80	5—03	0—09	4—39	1—16	2—59	0—53	0—08	1—01	.....	.....	15—51	Light. N. W.	Chop.	7	
	19	3 630	4—51	12.47	5—55	0—21	3—58	1—10	2—29	1—13	0—43	1—00	.....	.....	15—45	Light. V'riable	Smooth	6	September 28th, 29th and 30th, worked on the Main Ship Channel.
Oct.	1	1 761	2—26	12.06	2—26	.....	2—23	1—02	0—58	1—02	.....	.....	.....	.....	7—51	Stormy. E.	Rough.	3	One load on Main Ship Channel. Started for Gedney's Channel 12.09 P. M.
	2	3 682	4—18	14.27	4—51	0—10	4—09	1—14	2—46	0—58	0—23	0—46	.....	.....	14—44	Light. V'riable	Swell.	6	
	3	3 069	4—12	12.13	4—22	0—04	3—30	1—02	2—06	1—18	0—06	0—50	.....	0—50	13—58	High. S. E.	Swell.	5	
	5	2 370	3—17	12.03	3—24	0—07	2—40	0—48	1—35	1—06	0—20	1—00	.....	4—00	14—53	Light to Strong. S. E.	Heavy. Swell.	4	Delayed at anchor by fog.
	6	3 766	4—29	14.00	5—04	0—16	3—55	1—04	2—43	0—54	0—19	0—35	.....	.....	14—15	Fresh. N. W. to N. E.	Chop	6	Oct. 7th, S. E. storm —wind and rain— no work.
	8	2 088	3—11	10.93	3—35	0—19	2—29	0—42	1—48	1—03	0—35	0—48	.....	4—00	14—55	Fresh. N. W	Heavy. Swell.	4	Delayed at anchor by weather.
	9	3 690	4—44	13.00	4—59	0—15	3—31	1—07	2—12	1—28	.....	0—46	.....	.....	14—03	High. N	Swell.	6	
	10	2 216	3—14	11.42	3—26	0—08	2—17	0—56	1—09	1—00	0—04	0—57	.....	.....	9—45	High. N	Swell.	4	
		102 353	141—46	12.03	163—07	10—02	99—22	30—22	62—35	32—13	18—31	22—55	2—24	17—18	448—00	.....	.....	175	

the amount estimated to be necessary to attain the desired results by the only other project presented. During an absence of Colonel Gillespie the improvement was in charge of Colonel McFarland. After his demise Colonel Gillespie again took charge, and under him, and in accordance with his plan and recommendation, the improvement was completed.

As was to be expected, in an undertaking of such a nature and magnitude as this improvement, various difficulties had to be encountered and overcome, by the engineers in charge as well as the contractors; in view of which it is worthy to mention that, from the commencement to completion of the work, the most harmonious relations existed between them; the representatives of the Government being always considerate of the contractors, they, in turn, co-operating to bring the undertaking to a successful consummation.

The Pumps.—As the pumps are the chief features of the plant, it can be readily understood that, substantially, as work the pumps so works the plant. Though a self-propelling and self-containing dredging steamer needs to be in all its appointments rightly constructed and properly managed, yet it is apparent that however well everything else may be contrived, and however skillfully managed, the results attained will be but partial without an effective pump. Besides, as the pumps are the parts of the plant submitted to the greatest wear by the action of sand, and are liable to the greatest number of breakages from various resistant objects passing through them, they need to be provided with the best possible means of endurance and ready repair. That the pumps employed on this work meet all these requirements is sufficiently demonstrated by what has been done with them, not only on the improvement of New York Harbor, but by what they have accomplished on difficult work elsewhere. So well contrived to resist breakage and so powerful are they, that they have drawn up from a depth of 35 feet under water, and elevated, from the bed of the channel to the mouth of the discharge pipe, a height of 46 feet, bars of pig iron heavy as a man can lift, cannon balls, etc., and with no damage to themselves requiring more than twenty minutes to repair. Without special provisions for encountering such obstacles, they would often have totally wrecked the pumps.

Pump Dredging.—Pump dredging can be employed where no other system of dredging is possible, as shown by its application to the improve-

ment of New York Harbor, on which the steamer dredges have successfully worked in a heavy seaway. It leaves a smoother and more uniform bottom than any other system. It can do the work at much less cost than any other system of dredging, wherever it is applicable. It can go over the ground much more rapidly where only a slight cut of material is required to be taken, and it is the only system of dredging that can be employed to keep channels open that have a tendency to fill up, as a dredging steamer can pass over large areas and take only the needed skimming of material.

Reasons for Improving Other Channels.—As the improvement by widening and deepening Gedney's and Main Ship channels has been successfully accomplished at an expense to the Government not exceeding 23½ per cent. of the originally estimated cost, and as the improvement of these channels is proven to be permanent, more and better reasons than before now present themselves to show why the Swash Channel also should be improved.

The Permanence of these Improvements.—When the Government concluded to make appropriations for the improvement of New York Harbor, so much doubt existed on the part of its engineers as to whether it could be accomplished by dredging, both because of the uncertainty of the permanency of the work and the difficulty of its execution, that they concluded the best project for accomplishing the improvement of Gedney's Channel and that portion of the Main Ship Channel lying east and west, would be to contract the flow of water between Coney Island and Sandy Hook, by constructing a half-tide or higher stone dike from Coney Island, 4 miles or more toward Sandy Hook; and to dredge that part of the Main Ship Channel lying north and south, at an estimated cost of from $5 000 000 to $6 000 000.

Owing, however, to the first appropriation ($200 000) being stipulated for dredging in Gedney's Channel, dredging in this channel was commenced, but with little expectation on the part of the Board of Engineers that the results would be permanent. Careful comparative surveys made at a later time to determine the question of permanency, showed that so far as the work had been done the depth of the channel was fully maintained, whereupon dredging was continued. Other surveys, for the same purpose, from time to time, were made, which further confirmed the same satisfactory results.

In every instance test surveys show that the increased depths of the

channels are fully maintained and even increased. Hence it is safe to conclude that the Swash Channel, were it similarly improved by the same means, would also maintain its depth and width. Other reasons for this conclusion are:

First.—The Swash is not exposed to sedimentary deposits from the Raritan Bay as is the lower portion of the Main Ship Channel.

Second.—It is a shorter channel-distance from the upper part of the Main Ship Channel to the ocean, and, therefore, would be more likely to be scoured by the water from the Narrows, by way of the upper part of the Main Ship Channel than would be the lower section of the latter channel.

Third.—It is a more direct course between the upper part of the Main Ship and the Gedney's channels than is the Main Ship from its upper union with the Swash to the Gedney's Channel.

Fourth.—Surveys of the various channels from as far back as 1835 show that the Swash is in as good condition as it was 56 years ago.

Fifth.—The velocity of the flow of water through the Swash is twice as great as it is in the Main Ship Channel on the Knolls, and its flow is nearly parallel with its axis (the same is true of the Gedney's Channel), which also facilitates navigation, while part of the tide crosses the axis of the Main Ship Channel.

In confirmation of these statements the following is taken from Colonel McFarland's report for 1886:

"A comparison of the survey made by Major G. L. Gillespie, Corps of Engineers, in 1884 (see annual Report of the Chief of Enginers for 1885, pages 776 to 785) with those previously made, shows that the changes which have occurred in the channels since 1835 have been very slight. The 24-foot curves of the Main, Swash, East and Fourteen-Foot channels have all moved slightly westward and southward, but still embrace about the same area of shoals. But while, in 1835, bars with less than 24 feet on them extended completely across Gedney's, the Main Ship and the Swash channels, these bars, at the time of Colonel Gillespie's survey in 1884, had nearly disappeared, leaving in each case only spots where there was less than 24 feet of water. For fifty years, then, it will be seen that the natural tidal scour, which is strong enough at the Narrows to maintain a channel a mile wide, with depths of 100 feet in it, has been only able below the Narrows to maintain a 30-foot channel, with two spots in it where there is 24 feet depth.

"From current observations taken by Colonel Gillespie in 1884 it appears that a good deal of the ebb volume from the Narrows passes to the northward of the south side of the Swash Channel and that a large

part of the Raritan Bay ebb volume passes between the south side of the Knolls and Sandy Hook, leaving a wedge-shaped area at their junction near the Knolls, over which the rate of the current is reduced about one-half and where a part of the current passes directly across the channel instead of along its axis.

"This is also true of the flood tide; on the flood there is the same diminution of rate across this area, and a part of the tide crosses the channel at right angles. Some of the velocities observed by Colonel Gillespie in 1884 in these channels are as follows:

Localities.	Maximum ebb (miles per hour).	Maximum flood (miles per hour).
In the Swash Channel, abreast of the Roemer Beacon.....	1.60	1.20
In Main Ship Channel, on the Knolls.....	0.80	0.65
Close off the point of Sandy Hook, and due north of it.....	2.22	1.70 to 1.20
In the deep hole halfway between Sandy Hook and Gedney's	2.00	1.50
In Gedney's Channel.....	1.60	1.30

"Both in the Swash and in Gedney's Channel the ebb current is nearly parallel with the axis of the channels, and from the measurements so far taken the relative ebb velocities in both channels appear to be the same, but the flood in Gedney's Channel appears to exceed that of the Swash Channel by 0.09 miles per hour.

"In the spring of 1872 Brigadier-General (then Lieutenant-Colonel) John Newton, Corps of Engineers, had a series of current observations taken in the lower bay, with the following results:

Place.	Surface velocity in miles per hour.	Bottom velocity in miles per hour.
Fourteen-Foot Channel.....	2.20	0.40
East Channel.....	1.65	0.90
Swash Channel.....	2.00	0.40
Main Channel, close to and north of the point of Sandy Hook.....	2.40	0.80

"These observations were stated to have been taken at the maximum velocity of the ebb tide."

From these tables it is seen that the flow is more rapid in the Fourteen-Foot than in the Swash Channel, and 100 per cent. greater in the Swash than in the Main Ship Channel; showing the tendency of the water to take the course of the shortest distance from the upper part of the Main Ship Channel to the ocean.

The question of improving the Swash Channel does not imply the substitution of the Swash for the Main Ship Channel, for were the

former channel improved and employed, both ends of the latter channel would still be used in conjunction with the Swash Channel. Considering the Main Ship and the Gedney's channels, as now improved, to be one and the same channel extending from the Narrows to deep water of the ocean, it will be seen that while its course from the Narrows to Flynn's Knoll is direct, and not seriously indirect from Flynn's Knoll to the ocean, yet its course from the Narrows to the ocean is very disadvantageously indirect.

By referring to the map, it will be seen, as Colonel McFarland says in his report for 1886, that:

"The Swash Channel is really a cut-off from the Main Ship Channel, leaving it about 6 miles below the Narrows, and joining it again at the eastern end of Gedney's Channel. The distance from the Narrows to 30-foot soundings outside the bar by the Main Ship Channel is 15 miles and by the Swash Channel 11 miles."

Hence from the Narrows to the ocean the distance is 4 miles less by way of the Swash than by the Main Ship Channel. The directions of the Beacon ranges of these two channels, according to the Government chart, shows that the deviation of the Swash Channel from the course of the Main Ship Channel, at their upper union, is 54 degrees; and at their lower union the deviation of the Swash from the course of the Main Ship Channel is 61 degrees; the sum of these two angles being 115 degrees; while the sum of two angles formed by the Beacon ranges in their course around Flynn's Knoll is also 115 degrees, being 50 degrees and 65 degrees, respectively. Hence from the Narrows to the inner mouth of the Gedney's Channel, the amount of deviation of the Beacon ranges from a straight course is the same, and the number of angles or turns is the same, by way of both channels. And there is not a great difference between the magnitude of the several angles, the greatest being 65 degrees and the least being 50 degrees, both in the Main Ship Channel. Colonel McFarland, referring to these angles in his report for 1886, says:

"At the lower end of the Swash Channel it is necessary to make quite a sharp turn to the northward in order to get into the Gedney's Channel, and the turn from the Main Ship Channel into the Gedney's Channel is even shorter. More room should be given for vessels at these points."

Considering the extent of ocean traffic and travel and its rapid increase, it would seem, for convenience and safety, that, that portion of

the Main Ship Channel which turns upon itself to the extent of 115 degrees around Flynn's Knoll, should be relieved of a portion, if not all, of the larger ocean vessels ; not only because of the greater distance, inconvenience, expense, loss of time and danger attendant upon its navigation, but because of its less capacity than would be that of the improved Swash Channel. The great length and draught of many steamers require so large a turning area, that in doubling Flynn's Knoll or in following any short turns of the channels they are liable to swerve from one side of the channel to the other, causing increased liability of collision and grounding, as well as diminishing the capacity of the channel. In short, the relative safety and capacity of and the convenience and speed of passage through a channel of given width and depth, are commensurate with its straightness and inversely with its length. Besides, the current in the Swash Channel runs in line with its axis, while in that portion of the Main Ship Channel lying north and south the current is transverse to the axis of the channels, which diminishes the facility of its navigation. Again, as a great majority of the vessels entering and leaving the port pass through the Swash Channel, and as in passing out of this channel at either end their course is in a direction diagonal to the axis of the Main Ship Channel, and therefore to the course of the larger ocean steamers in their passage through the latter channel, it is evident that this is a source of liability of collision that would be obviated if all vessels were running in the same general direction.

Though the items of extra fuel and time, required by the indirection of the source of and the greater distance by, the Main Ship Channel, may be considered of small account, nevertheless, in view of the increasing number of large ocean steamers navigating these channels and the interest felt in saving time, especially with that class of steamers that are now obliged to employ the Main Ship Channel, they are not to be overlooked. The saving on these two items of fuel and time, to say nothing of other advantages, would aggregate a sum, in course of time, sufficient to pay the cost of amplifying the Swash Channel to a width of 1 000 feet and a depth of 30 feet at mean low water. An evidence of the advantages of the Swash over the Main Ship Channel is the fact that it is employed by nearly all, if not all, navigators who are not compelled to take the Main Ship Channel because of great draught of their vessels.

The Main Ship Channel, being a trifle deeper as well as somewhat wider, than the Swash Channel, it has been employed by the larger

steamers, and the use of it by these deep draught vessels has had the effect of keeping it open and increasing its depth. But as the current in the Swash is twice as rapid as in the Main Ship Channel, and in line with instead of across its axis, had the bed of the Swash been disturbed and stirred up in the same manner, it is possible that the depth of the Swash would now be equal to that of the Main Ship Channel before its recent improvement.

In his report for 1885 Colonel Gillespie says:

"The change in the character and draught of the vessels using the harbor during the past thirty years, particularly in the last ten years, has had its influence, no doubt, in slightly increasing the depth from 21 to 24 feet on the bar."

Why the Main Ship Channel between the upper and lower ends of the Swash, instead of the Swash Channel itself, was improved, may have been, presumably, for the reasons that: At the time the Government concluded to improve New York Harbor its engineers were not clear as to the best project to present or the best course to pursue, not knowing without trial what would be the result of dredging as to permanency; therefore they felt their way, step by step. Had they known at the outset that the entire work could be done by dredging alone, and had they then known, as they now do, that the results of dredging would be successful and permanent, possibly they might have concluded that the Swash was the preferable channel to improve first.

The project of improving this channel can now be viewed in the light of facts and experience which did not previously exist.

DESCRIPTION OF PLATES.

CIII. Chart showing the channels in the Lower Bay.

CIV. View of the dredging steamer *Reliance.*

CV. "Sheer-plan" of the after part of the *Reliance*, showing the arrangement of the drags and suction pipes.

CVI. Plan of main deck of after part of the *Reliance*, showing the arrangement of the boilers, propelling engines, main pumping engines, dredging pumps and auxiliary machinery.

CVII. Sections of the steamer between propeller engines and boilers, looking forward and aft.

CVIII. Details of the drags.

CIX. General plan of a dump scow.

CX. Longitudinal section of the after part of the *Reliance*, showing machinery, pumps, etc., and one of the bins for dredged material.

This paper, for special reasons, is printed without time being given for discussion, and discussions may be sent in for printing at a later date.

IMPROVEMENT OF NEW YORK HARBOR, NEW YORK,

IN CHARGE OF

LIEUT. COL. G. L. GILLESPIE,

CORPS OF ENGINEERS, U.S.A.

MAP OF THE

LOWER BAY

Showing

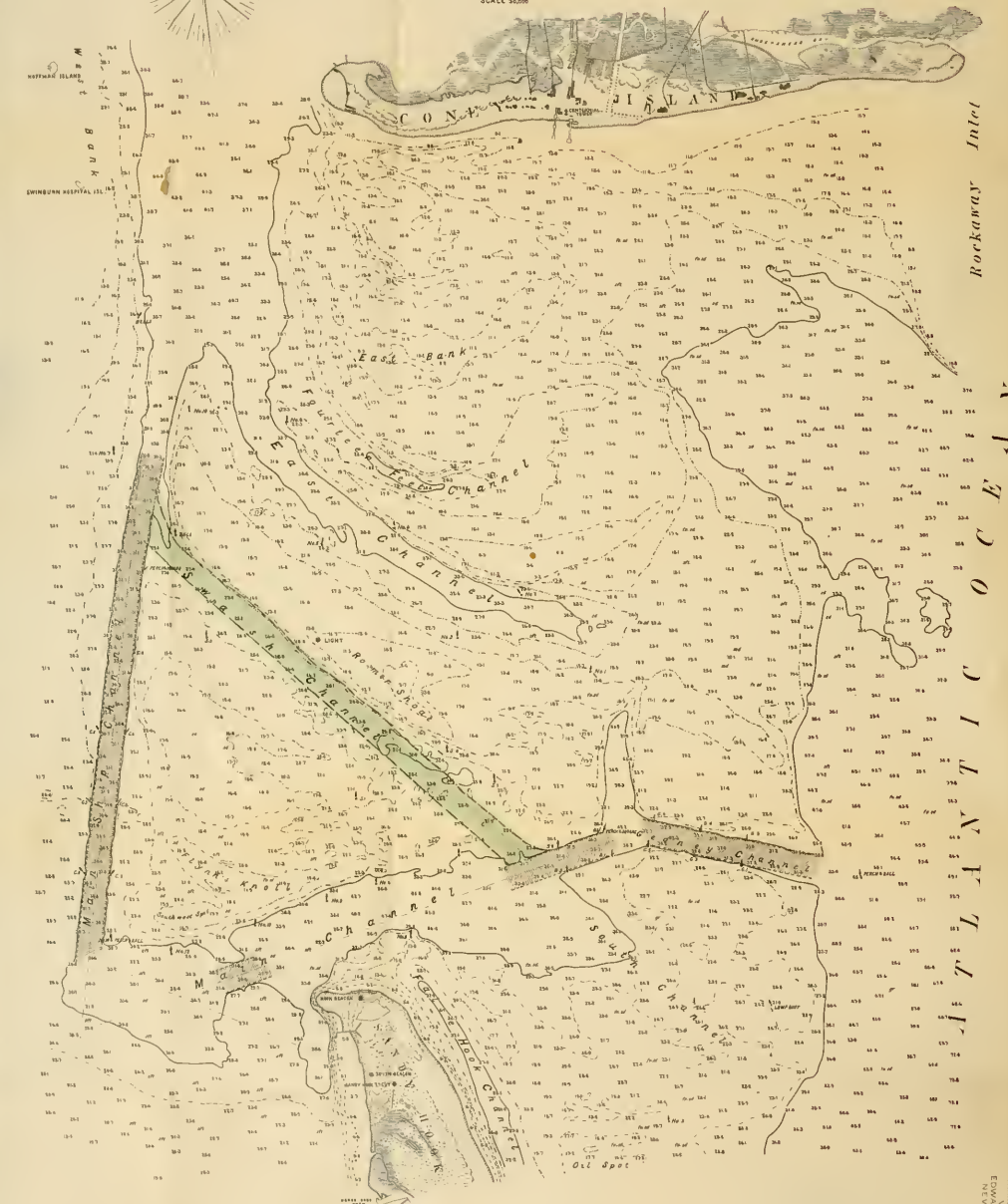
the Improved Channels dredged to 30 feet, mean low water, completed October 1891.

SCALE 1/50000

ENGINEER OFFICE, U.S. ARMY.
New York, N.Y., Dec. 2, 1891

This Map is compiled from the U.S. Engineer survey of 1884, supplemented by additional surveys of the improved channels made in June 1891

G. L. Gillespie
Lieut. Col. of Engineers U.S.A.



TIDES.

		Scale of Mean Dock
High Water	Mean interval after preceding	7h 29m
Low "	Transit of the Moon	13h 44m
Mean Rise and Fall of Tides		4.8 feet
" " " "	Spring Tides	5.6 "
" " " "	Neap Tides	4.0 "

Scale of feet

Scale of miles

SOUNDINGS.

Soundings are expressed in feet and tenths and refer to Mean Low Water (U.S.C.S.) at Horseshoe Dock

The 6 feet curve is shown thus

- 12 " " " " "
- 16 " " " " "
- 24 " " " " "
- 30 " " " " "

The proposed improvement of the SWASH CHANNEL

PLATE LIII
TRANS. AM. SOC. C. ENGRS.
VOL. XXV. No. 513
EDWARDS ON DREDGING
NEW YORK HARBOR

PLATE CV
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VOL XXV Nº 513.
EDWARDS ON DREDGING
NEW YORK HARBOR.

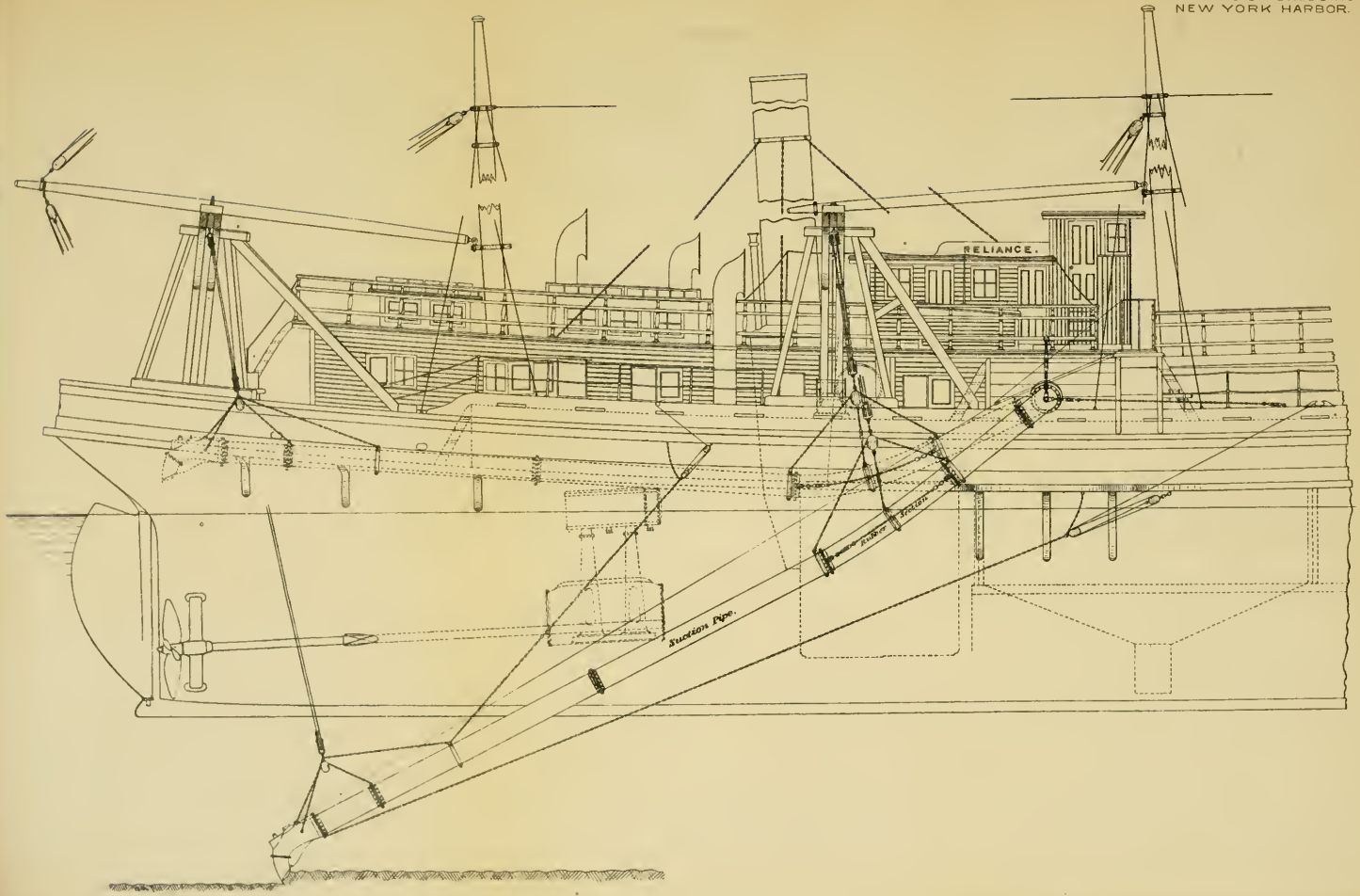

PLATE CVI
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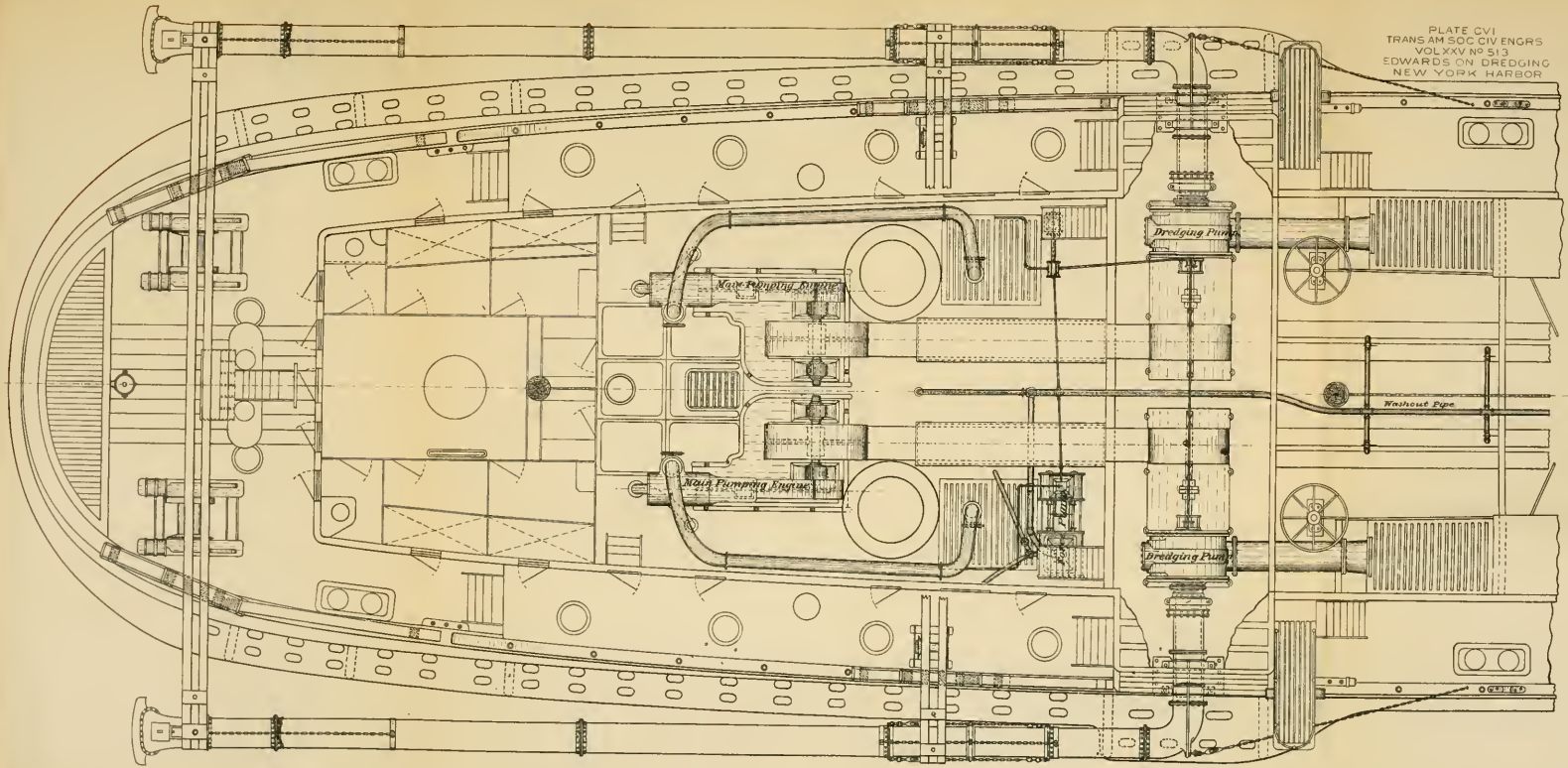

PLATE CVII
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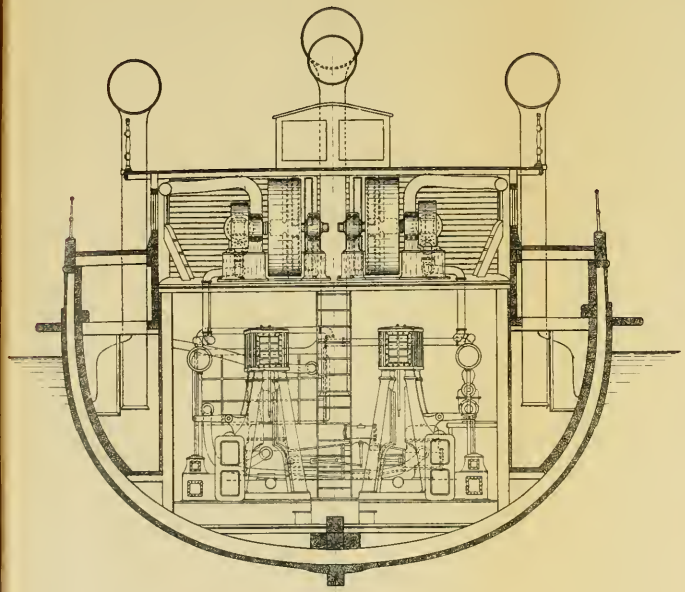

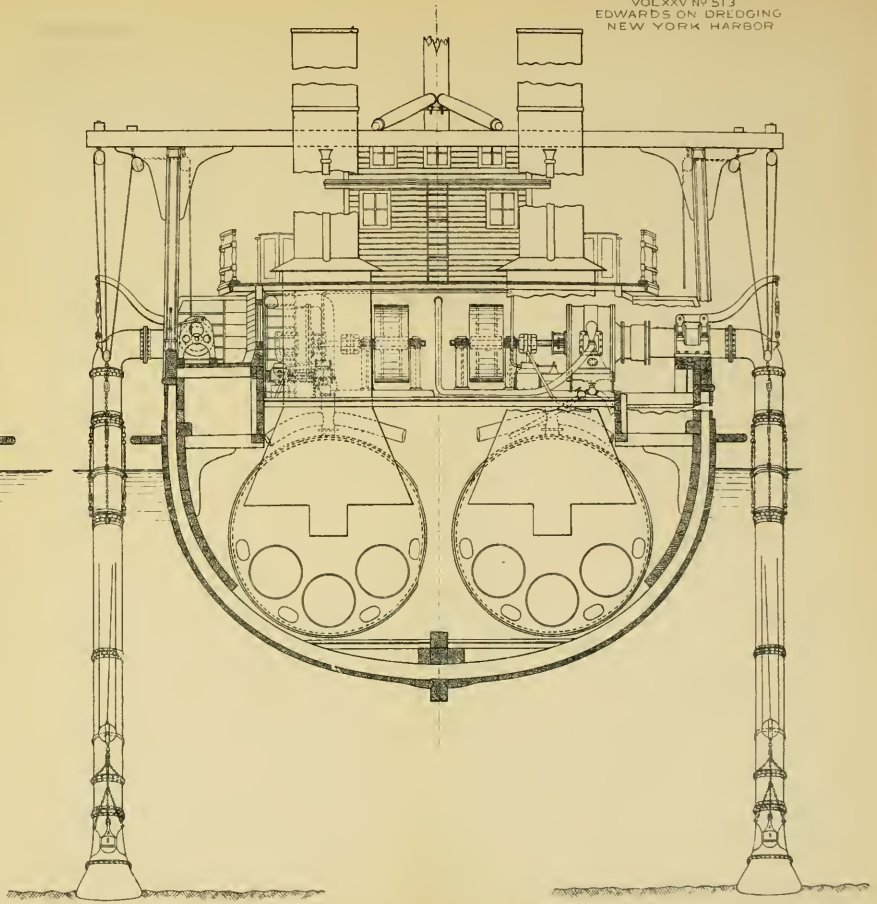

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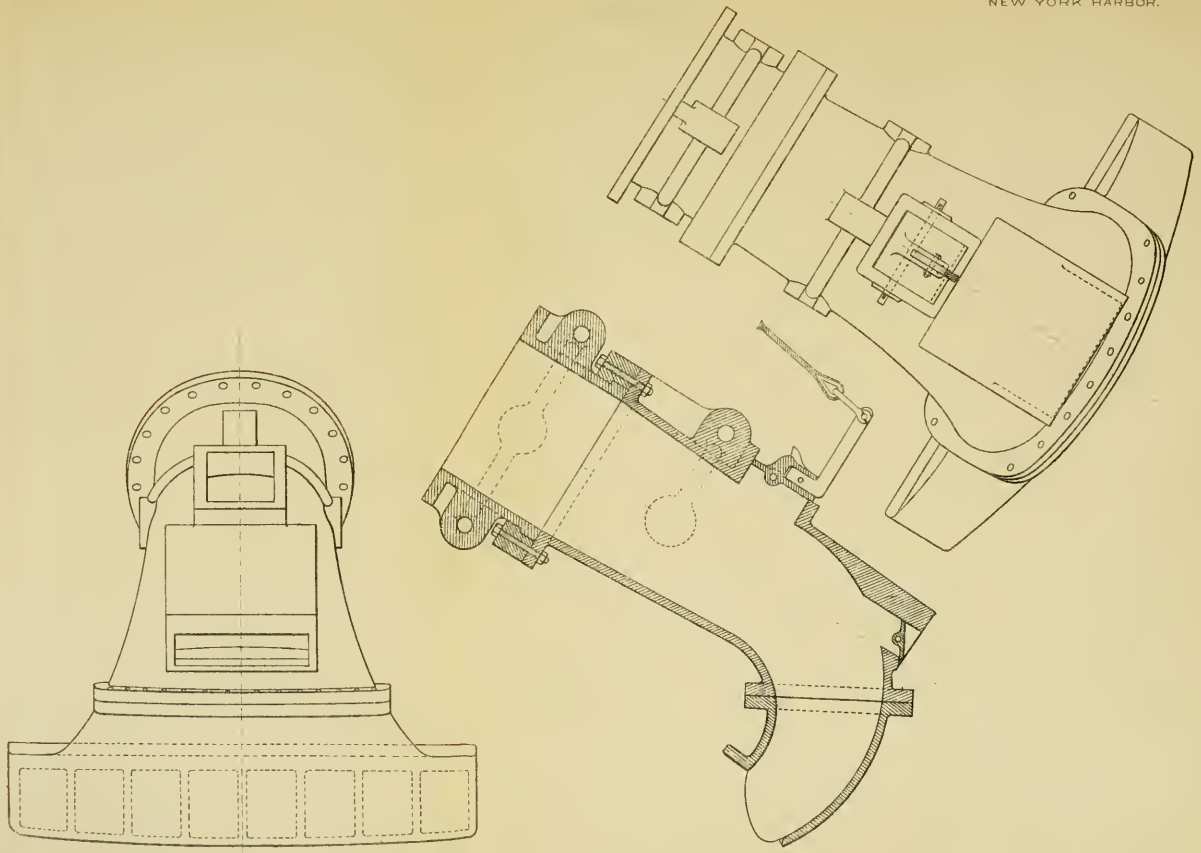

PLATE CIX
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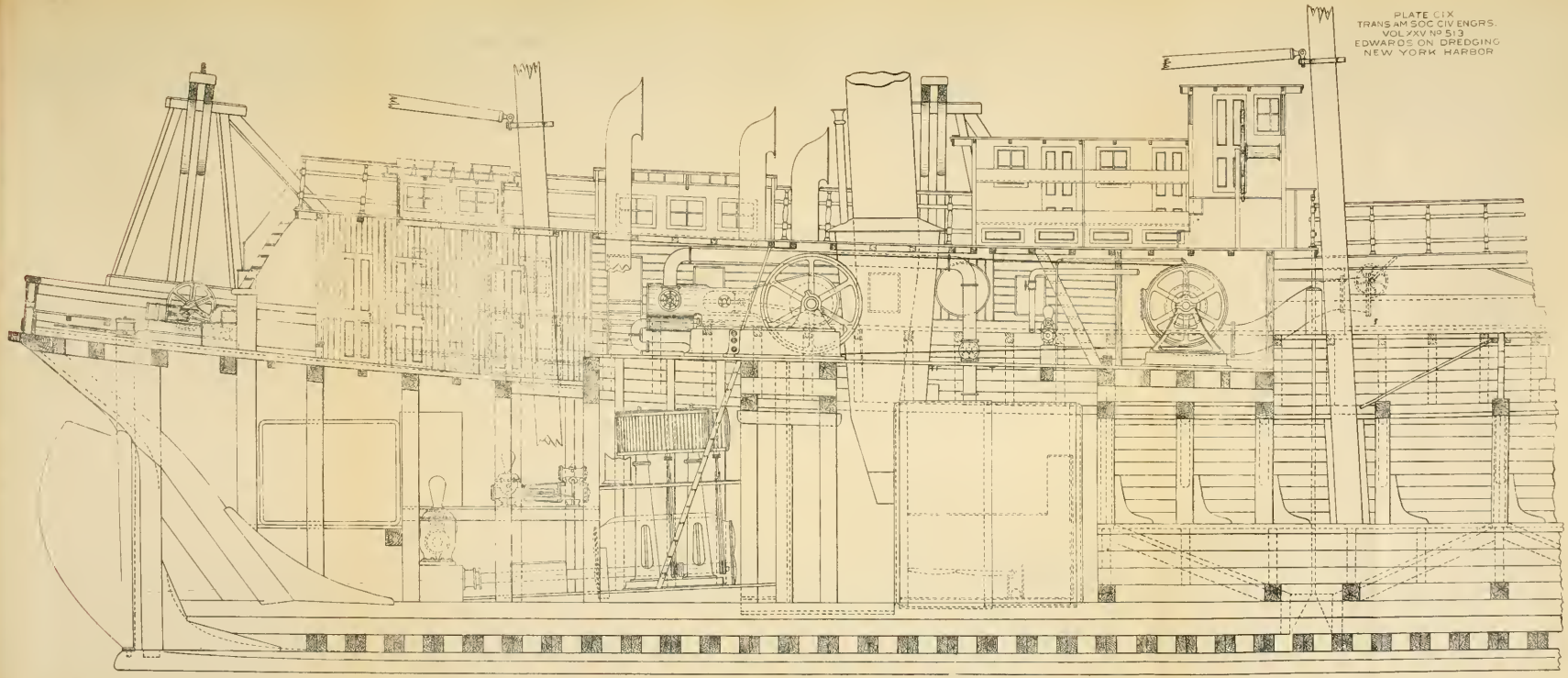

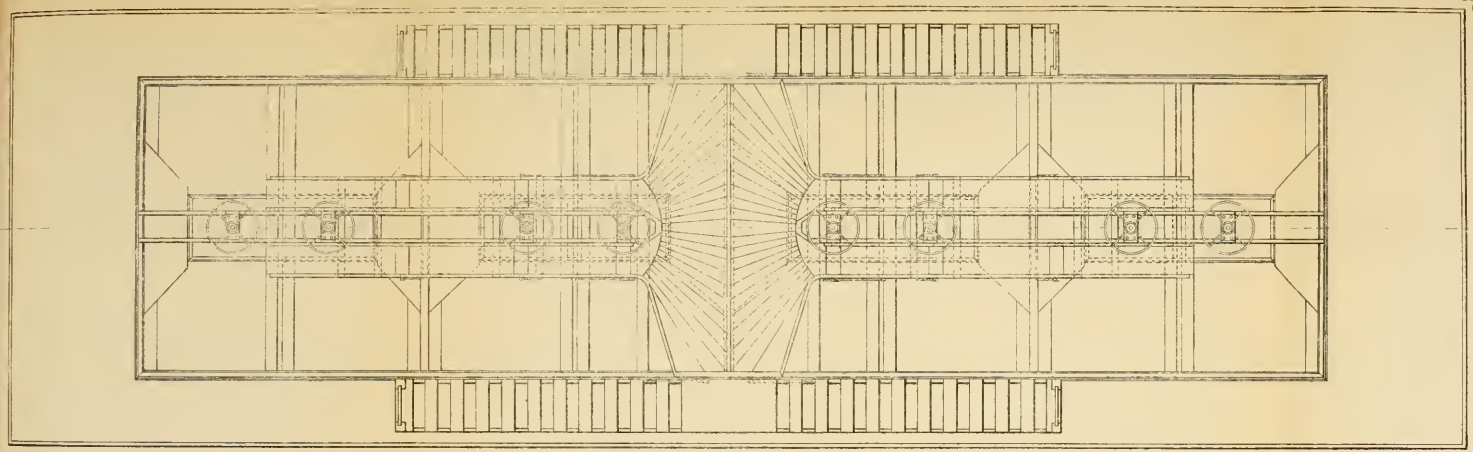

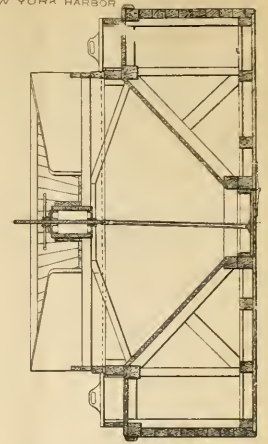

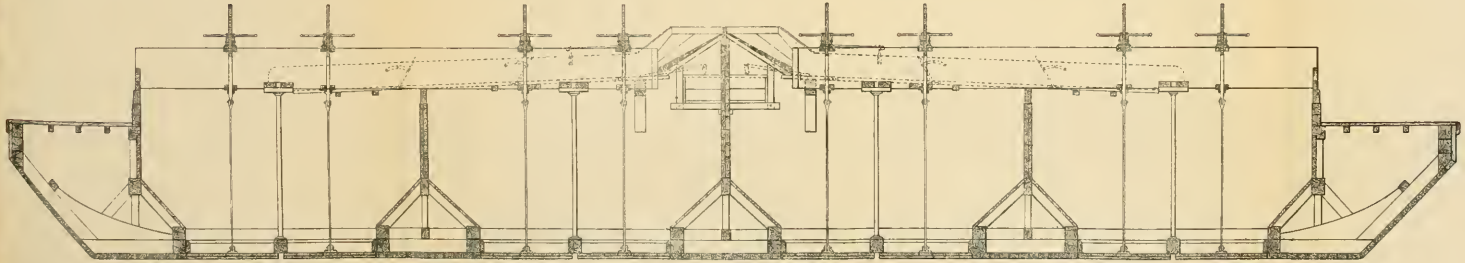

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514.

(Vol. XXV.—December, 1891.)

LONGITUDINALS vs. CROSS-TIES FOR RAILWAY TRACKS.

By E. E. RUSSELL TRATMAN, Assoc. M. Am. Soc. C. E.

WITH DISCUSSION.

The following remarks were suggested by the announcement of the title of the paper by Mr. T. C. Clarke on "The Advantages of a Longitudinal Bearing System for Railway Tracks," but those dealing with the subject generally, were in the main originally prepared before the writer had seen the paper, and when he did not know, therefore, that Mr. Clarke would confine his attention to metal longitudinals. The remarks referring specially to points in his paper were, of course, written after a careful perusal of that paper. The general opinions of railway engineers in countries where the continuous bearings of longitudinals and the separate bearings of cross-ties have both been tried, appear to be decidedly in favor of the cross-tie system for ordinary purposes, although the longitudinals continue to be quite extensively used under special conditions, more especially across bridges.

The question of the use of wooden longitudinals presents some points of interest, and will be considered first. In England, the Great Western, the London and Northwestern, the Lancashire and Yorkshire, the

Midland, and other railways, at one time used longitudinal timbers, but the system has been generally abandoned, except in some cases for bridge tracks. The Great Western Railway still maintains the longitudinal system upon its main line of broad and mixed gauge between London and the west of England, and upon the standard gauge line between Swindon and Gloucester, but does not use it on other parts of its lines, or construct any new track of this type. The rails are of bridge section, weighing 68 pounds per yard, and are secured by fang bolts which pass through the flanges. The rails do not rest directly upon the longitudinals, but upon a continuous line of yellow pine shims about 6 inches square and 1 inch thick. The longitudinals are creosoted timbers 15 inches wide, 7½ inches deep on the outer side and 7 inches deep on the inner side. They are connected by transverse timbers about 6 x 6 inches at intervals of 11 or 12 feet. This is a remarkably easy riding road, especially in the modern cars on four-wheel trucks.* The broad gauge lines are to be converted to standard gauge in May, 1892. Mr. Lambert, the General Manager of this railway, has informed the author that wherever it becomes necessary to relay any portion of the old broad gauge track the new standard pattern of bull-head rails is being adopted, but there is no intention of abandoning the old longitudinal system when the gauge is altered, where the track is in good condition and does not require renewal. Of course, gradually, as the old track becomes worn out, it will be replaced by the modern pattern;† but it will be a long time before the whole of it has disappeared from the Great Western Railway system.

The comparative costs, at approximate English prices, per mile of single track, including tracklaying but exclusive of ballast, for longitudinal and cross-tie systems, are about as follows:

Great Western Railway.—Bridge rails on longitudinal timbers	$9 440 00
London and Northwestern Railway.—Bull-head rails in cast iron chairs on wooden cross-ties	10 475 00
West Riding, Hull and Grimsby Railway (before acquired and relaid by Great Northern Railway).—Flange or tee rails on wooden cross-ties	8 640 00

* This track was described and illustrated by the author in *Engineering News*, New York, February 21st, 1891.

† Described and illustrated in the author's paper on "English Railway Track," *Transactions* American Society of Civil Engineers, June, 1888.

At a meeting of the Institution of Permanent Way Inspectors, England, in 1887, it was stated that several advantages were gained by the use of the longitudinal bearing. The rails do not have to act as girders, and may therefore be very considerably reduced in weight, and being generally of the bridge or flange sections, cast-iron chairs are not required. It is safer in case of derailment, as the wheels may run, and have run along the longitudinals until the train has been stopped. On the other hand, the larger scantling of timber required makes it more expensive per foot, and more awkward in repairs, while to replace one timber the whole rail length must be taken up. The trackmen must have some practical and technical knowledge of carpentry as well as of ordinary track-laying, and the work is rather complicated, especially at frogs and switches. It is difficult to keep the drainage perfect, as it is confined between the longitudinals. In a report on a derailment accident on the Great Western Railway, in 1889, one of the inspectors of the Board of Trade stated that it is difficult for the trackmen to know if longitudinals are thoroughly tamped until they watch an engine passing over the places that have been under repair, as the stiffness of the rail and the longitudinal keeps the latter level, even when it may not be properly packed underneath with ballast.

The following are the quantities of timber per mile of single track on different classes of track:

	Feet, B. M.
(*A.*) English; cross-ties 9 feet x 10 x 5 inches, uniform spacing of 3 feet, 10 ties to a 30-foot rail = 1 760 ties per mile	66 000
(*B.*) English; cross-ties 9 feet x 10 x 5 inches, closer spacing at joints, 12 ties to a 30-foot rail = 2 112 ties per mile	79 200
(*C.*) American; Boston and Albany Railroad, cross-ties 8 feet x 7 x 8 inches, uniform spacing 2 feet center to center, 2 640 ties per mile	98 560
(*D.*) American; cross-ties 8 feet x 6 x 8 inches, 2 800 ties per mile	89 600
(*E.*) English; Great Western Railway, 7 feet gauge, longitudinals 15 x 7½ x 7 inches, with 480 transoms 6 feet x 6 x 6 inches, at intervals of 11 feet	104 340
(*F.*) Suggested plan; longitudinals 12 x 7 inches, with 352 transoms 49 x 6 x 6 inches at intervals of 15 feet	78 232

The latter plan represents the longitudinal system reduced to a

minimum, and rails with a base less than 6 inches wide would probably cut and split the timber unless shims or tie-plates were used.

In nearly all the writings on this subject great stress is laid upon the difficulty of effecting proper drainage of the roadbed, but it seems to the author that this difficulty is made to appear too great. It is mainly a question of ballast, and there would probably be considerable trouble with sand, earth, or other close ballast. But with broken stone, or even coarse gravel, if free from clay, there should be much less trouble, although the latter might certainly tend to pack into a hard mass to a certain depth. With longitudinals laid upon a bottom course of large broken stone, and well ballasted and packed with broken stone of the ordinary size, there should be ample drainage, even for a sudden heavy rain-fall.

The use of longitudinal timbers to carry the rails across bridge floors is quite extensively adopted in other countries. The two tracks across the great Forth Bridge, Scotland, are laid with rails of bridge section weighing 120 pounds per yard, and secured to teak longitudinals by screw spikes 8 inches long, which pass through the flanges of the rail. There are 20 of these spikes to each rail. The longitudinals are about 12 x 6 inches in section, and are kept in line by horizontal and vertical wedges driven between them and the sides of the iron "trough" in which each line of rails lies, and which prevents the use of transoms or other transverse connections to maintain the gauge. The timbers rest upon a continuous bearing of wooden blocks with pitch filling. There is no ballast. The track and floor seem to be too rigid, needing some means of absorbing the vibrations. The trains make a harsh metallic sound and a jarring in running across the bridge, and these are unpleasantly noticeable in riding in the cars. The change is very pronounced when the train runs onto the embankment approach, where the track is laid with the same rails on longitudinal timbers, with transoms, in broken stone ballast, the train then riding smoothly and quietly. The Severn Bridge, England, is a single track structure, 3 385 feet long, with 21 through spans. The track is laid with rails of bridge section secured by fang bolts and screws to creosoted pine longitudinals, 14 x 7 inches in section, connected at intervals of 11 feet by timber transoms. Guard rails of angle iron $4\frac{1}{2} \times 4\frac{1}{2}$ inches are secured to the longitudinals along the inner side of the rails, bearing against the rail flange, and leaving 2 inches clear between the rail and

guard. This track was described in the paper already referred to on "English Railway Track," *Transactions* Am. Soc. C. E., June, 1888. The London and Northwestern Railway, England, has bridge rails on longitudinal timbers across a viaduct near Stockport. A peculiar method of continuous bearing for rails is that used by the Midland Railway, England, across some short span bridges. Each rail is carried by two continuous angle bars, bolted to the rail and to the longitudinal timbers upon which they rest.

It is a very general practice on English railways to lay longitudinal timbers across bridges, generally without ballast, but these do not give the rail a continuous bearing, as the cast iron chairs carry the bull-head rails clear of the timbers. This kind of track, consisting of bull-head rails in cast iron chairs secured to longitudinal timbers laid on a solid floor of transverse iron troughs, is to be used for the elevated electric railway at Liverpool, England. This railway is a plate girder structure, with through spans, and a floor of transverse troughs, and the longitudinals will be kept in position by brackets riveted to the tops of the troughs. In India and the colonies, however, where flange rails are extensively used, it is a very general practice of the English engineers to lay these rails on longitudinal timbers across bridges. Longitudinal timbers across bridges are also used on railways in France and other European countries, the tracks consisting sometimes of double-headed rails in chairs, and sometimes of flange rails resting on the longitudinals and secured by bolts or screw spikes passing through the flanges or merely bearing upon them. Angle iron brackets riveted to the bridge floor may be used to keep the longitudinals in position. The Victoria tubular bridge which carries the Grand Trunk Railway across the St. Lawrence River at Montreal, has a single track laid with flange rails spiked to longitudinal timbers which rest on the transverse I-beams of the floor. Mr. Hannaford, M. Am. Soc. C. E., Chief Engineer, states that the spikes have reversed points. The question of the relative merits of longitudinals or cross-ties for carrying the rails across bridges was discussed at the International Railway Congress held at Paris in 1889. The conclusion arrived at was that both systems possessed certain advantages and were used according to local conditions, and that neither could be definitely recommended to the exclusion of the other. In Fig. 1 is shown the floor system of the double track bridge carrying the Ceinture Railway (or Belt Railway) of Paris across the

River Marne at Champigny. The longitudinals are held down at intervals by straps under the rails and vertical bolts. The floor is covered with ballast. In Fig. 2 is shown the floor system of the single track Gagniéres Viaduct, France, in which the longitudinals are laid against angle iron brackets riveted to the floor beams. These beams are below the top chords of the main girders, which thus form a high guard-rail.

In regard to metal longitudinals, the most extensive experiments with these has been in Germany and Austria; but while the use of metal cross-ties is being extended considerably, the longitudinals are only used to a limited extent on new work, and in some places they are being aban-

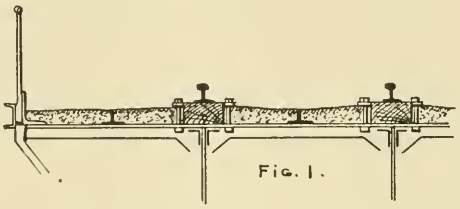

Fig. 1.

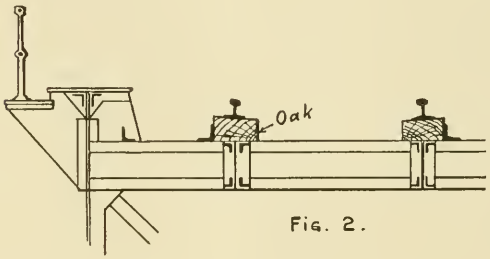

Fig. 2.

doned. The transverse connections generally give some trouble, especially at curves, and the general construction of the roadbed and the work of tracklaying require more care and labor, while the cost of maintenance has in some cases been found to be higher with longitudinal than with cross-ties. There has also been difficulty in maintaining the gauge. A German authority stated in an article in a technical magazine, in 1886, that with longitudinals of metal it is undesirable to have many transverse connections, as they will counteract the advantages of the longitudinals and introduce some of the disadvantages of the cross-ties. He recommended a connection at the ends of the longitudinals, and also at the middle if more were considered necessary. The continuous

bearing is theoretically the better, giving a uniform transmission of the wave motion under rolling loads, with a consequent smoother rolling and less wear. The separate bearings of cross-ties exert a disturbing influence on this wave motion, causing uneven wear, but it is possible to obtain sufficiently satisfactory results in practice in this respect from the cross-tie system. The continuous bearing gives a better resistance to lateral strains on the rails, but with cross-ties this can be provided for by the use of tie-plates. The quantity of ballast is about the same, but there is difficulty in draining it when longitudinals are used. Renewals are probably cheaper with cross-ties. After considering the various points he came to the conclusion that the cross-tie system will continue to be the standard. The same conclusion was also arrived at by a convention of the German Railway Union a few years ago.

Mr. Clarke, in his paper, does not seem to have done his subject justice in merely quoting the statements of Mr. Hohenegger concerning the latter's system of track on the Northwestern Railway, Austria, of which he is Chief Engineer. In the report of the author to the U. S. Department of Agriculture upon "The Substitution of Metal for Wood for Railway Ties" (1890), there are many other statements concerning metal longitudinals and the experience therewith, which show different views from those entertained by Mr. Hohenegger. A longitudinal system would require more care and more skilled labor in laying and maintenance than it is now usual to find on any but the first-class lines in this country, and the work would be especially complicated at frogs and switches and on sharp curves. A defect of this system of track is the very much smaller area of bearing surface upon the ballast as compared with that of the cross-tie system, and, as already stated, it is not considered desirable to increase the number of transverse connections. This, added to the increased difficulty in tamping, and other features, would tend to add largely to the maintenance expenses by continual work in keeping the track up to surface. Under heavy traffic it would probably not be found advisable to make any great reduction in the weight of the rail. For existing roads adopting this system the entire reconstruction of the track would be a difficult and costly work, especially upon lines with heavy traffic, and it is a question whether the efficiency and economy resulting from the new track would be sufficient to warrant the change. It is only upon lines with heavy traffic that the introduction of the longitudinal system of track is likely to be con-

sidered, even if its advantages are proved; but the traffic upon the lines mentioned by Mr. Clarke is comparatively light.

The following extract from the report just mentioned is the summary of the information therein presented in regard to longitudinal bearings, and in regard to the Berlin Metropolitan Railway, later information is to the effect that after six years of comparatively heavy traffic the iron longitudinals were abandoned for wooden cross-ties on account of the insufficient support across the track for bearing surface and for resistance to lateral strains upon the track:

LONGITUDINALS.—This type of track is only used to a limited extent, and, as shown by special reports and other information, its use is not increasing. The construction of the track is more difficult and requires more labor than that with cross-ties. Maintenance, renewals and repairs are less easily managed, and greater care must be paid to the ballasting and drainage. On the other hand, it has the advantage of giving the rails a continuous support, and, consequently, with a well-packed roadbed would make a very smooth riding track, but probably at a higher cost than an equally good track of another type. The construction of the roadbed and arrangement of the ballast involve considerable care and cost, as, owing to the difficulty of drainage, special means have to be taken, by the use of courses of large rough stone or drain pipes. to carry water away quickly. Longitudinals have been adopted for the city railways of Berlin, Germany, having been proved the best for reducing the noise of passing trains on this viaduct line connecting main lines of railway. It was originally thought that longitudinals would make a better and cheaper track, avoid shocks at the rail joints, and, by their long bearing in the ballast, require less work of maintenance. Also, that lighter rails could be used with the continuous bearing. It is, however, difficult to maintain the ballast so as to keep a continuous and even bearing inside the longitudinal. The economy in material effected by the use of longitudinals is reduced or negatived by the necessity of using transverse connections of ties or tie-rods to hold the track together and maintain the gauge. This system is awkward on curves, and renewals are difficult. Each piece must be bent hot at the works, or have the holes for the rail fastenings made to fit a certain radius of curve.

In regard to steel cross-ties, Mr. Clarke says: "The tie being but a shallow piece of channel iron with flanges 2 to 3½ inches deep has no stiffness crosswise." This seems to imply that there is but one form of cross-tie, but in point of fact they are of various forms and sections. The channel or trough section, of certain shapes, has been found in general the most satisfactory, and it is not difficult to stiffen the ties if found necessary. Many of these now in use, however, are sufficiently stiff to retain their shape under heavy traffic, and give good satisfaction. Mr. Clarke's remarks upon the necessity of secure fastenings, apply equally to fastenings for cross-ties.

On the whole, it appears to the author probable that the cost of a system of track with longitudinals, fitted for heavy traffic, would be considerably in excess of the cost of a track with cross-ties of equal strength and efficiency. It is true that some engineers are in favor of the longitudinal system, theoretically at least, and it is possible that it may constitute the track of the railway of the future; but its introduction does not seem probable, and the most important work now to be done is to provide for the improvement of the efficiency of the present system of track. These remarks apply to the track in general, but there may be special cases, as at bridges, etc., where the use of longitudinal bearings may be advantageous.

Granting that much of the existing track is defective, the author does not think there is room for claiming that (as stated in Mr. Clarke's paper) the system of track is giving out at all points, for the existing tracks are capable of very great improvement without departing from the present general system of construction.* The adoption of heavier rails is continuing, but the extra cost deters many companies from relaying their track even when it is admittedly too light for the traffic. In addition to the heavy rails mentioned in the paper above referred to, the single track of the St. Clair tunnel at Port Huron, Mich., connecting the Grand Trunk Railway and the Chicago and Grand Trunk Railway, is laid with rails of the Sandberg pattern, weighing 100 pounds per yard; the Boston and Albany Railroad has adopted a 95-pound rail; the Manhattan Railroad (New York) and the Chicago and South Side Rapid Transit Railroad (both elevated railways) are using 90-pound rails; the Baltimore and Ohio Railroad is laying 85-pound rails upon its Philadelphia and Baltimore Division; the Great Northern Railway has laid 80-pound rails on the heavy grades of the Pacific extension of the St. Paul, Minneapolis and Manitoba Railway, and the New York, Lake Erie and Western Railroad and the Lehigh Valley Railroad have adopted 80-pound rail sections. It may be noted also that the government of South Australia has adopted an 80-pound flange or tee rail for use on the main lines of its railways.

It is to the interests of the railways to employ skilled labor or at least intelligent labor on all work of importance, as cheap labor is dear labor in many cases. An instance of this came under the author's notice at

* This view was presented in his paper on "The Improvement of Railway and Street Railway Track," *Transactions* American Society of Civil Engineers, March, 1890.

Montreal last summer while watching the loading of new steel rails upon Grand Trunk Railway flat cars. The rails were in a triangular pile, and half a dozen men were engaged in lifting them by hand and throwing them on a truck or cart so that they struck forcibly against one another, sometimes falling on the head and sometimes on the side. When all the loose rails at the bottom of the pile had been thus loaded on the truck, the top of the pile was pushed over, about a dozen rails rolling over and over, and writhing and bending like snakes as they struck the ground. When the loading was completed the truck was driven around to the side of a flat car, and being of the same height, two iron bars were placed across as skids. The rails were then thrown onto the skids, striking on head, side or base, then slid across, lifted with rail tongs and placed or dropped upon the other rails on the car. Supposing similar methods to have been practiced in loading at the mills and into the ship, unloading from the ship and again in unloading on the track, it may readily be imagined that many of the rails were subjected to serious injury by this carelessness in handling.

DISCUSSION.

T. C. Clarke, M. Am. Soc. C. E., writes: Mr. Tratman brings up evidence to show that in the general opinion of railway engineers and managers, the longitudinal system gives a smooth, easy riding track, but that it is more expensive to construct and maintain than a cross-tie system. When both systems are of wood there is no doubt of the truth of this conclusion.

The fourteen years' experience of Herr Hohenegger with a metallic longitudinal system does not seem to show that that system is more expensive to construct or maintain than a metallic cross-tie system. My own views can be briefly stated. So long as we can get wooden cross-ties at any reasonable cost we should continue to use them and endeavor to remedy their defects. When their cost becomes prohibitory, we should consider more carefully as between metallic cross-ties and metallic longitudinals.

The defects of the present system are: 1st, decay of the ties. This can be remedied by preservative processes. 2d, Spreading of rails. This can be prevented by some form of chair or tie-plate held to the tie by four spikes. This also prevents a third defect—cutting of the rail into ties. The fourth defect is, in my opinion, not to be remedied by the

present joints of rails. Two discontinuous rails cannot be united by any form of splice which will give a perfect joint under our heavy and rapid trains. Either the connection will be loose and deflection will take place, or if tight enough to prevent that, expansion, not being able to force the rail ends together, will cause kinks in the track, as was seen on the Lake Shore Railway. We want to return to a compound rail, made in two parts breaking joints, the parts united by bolts and rivets in larger holes. The office of these bolts or rivets should be not to transmit shear, but to hold the two parts of the rail together, and the shear will be transmitted by one part sliding upon the other. Such a rail, being of equal strength and elasticity throughout, would practically be jointless, whether it rested on ties or on longitudinal bearers.

When the time comes to decide between longitudinals or cross-ties of metal, the question becomes one of giving enough bearing surface, for with either system we prevent decay and spreading of rails, and by the use of compound rails can also abolish low and battered joints.

The trouble with all metallic cross-ties—either those illustrated by Mr. Tratman in his Government report, or those that have been since invented—is, that from motives of economy the designers have not given them enough bearing surface. They are all insufficient in depth to transfer the load on the rails to their centers. Hence the whole length of the tie cannot be available for bearing. A wood cross-tie system, with 8-foot ties of 8 inches face, 2 feet between centers, gives 1.32 square feet of bearing per linear foot of rail, and this is the least allowed for our present loads, for the reasons so clearly pointed out by Dr. Dudley and Mr. Shinn. None of the metallic cross-ties in use give much over one-half a foot of bearing surface per linear foot of rail.

The merit of the longitudinal bearing system is that the bearing is all directly under the load, and where it does the most good.

J. Foster Crowell, M. Am. Soc. C. E.—Mr. Tratman has given us a great deal of information in this interesting paper as to the use in the past of longitudinal track supports, and the facts he cites are instructive in that they show what types have failed and why their undoubted advantages for certain purposes have been overbalanced. In regard to what may be termed the modern revival of the longitudinal system, however, he does not give us much detail, but contents himself with a résumé or summary of general information ; he leaves the question open as to whether or not it is practicable to devise a longitudinal system that shall retain the acknowledged advantage of continuous bearing, and yet be free from the objections which have practically driven it out of the field, generally speaking.

In considering the relative uses of the two systems we must not lose sight of the fact that the cross-tie track has been retained not on account of excellence but in spite of almost fatal defects, to remedy which an enormous concentration of thought and effort extending over many

years has been expended without removing them. It can only be regarded, then, as something to be tolerated until we can be supplied with something better.

We are, most of us, inclined, I presume, to share Mr. Tratman's conclusion that the introduction of the longitudinal system, in a radical and general sense, now seems improbable ; and yet it is difficult to state precisely any theoretical or practical objection to its use that cannot be offset by one against the other system, save two, which are, the greater first cost, and the greater time and effort required to make it effective and keep it so. The first is in itself no argument, though a potent factor, but the second is most important both as factor and argument. We can, in other words, readily secure with the cross-tie in all situations, on all road-beds, and in fact, in emergencies, even without a road-bed, what would be difficult and often impossible with the longitudinal, viz., immediate working efficiency ; and this whether we are dealing with new track recently laid or old track which has been washed out or otherwise impaired, or on a road-bed being ballasted in service, as so frequently happens, in this country at least. This is such a vital consideration that in most cases it would govern even if the two systems were otherwise balanced in merit and of equal cost; but its potency diminishes as the conditions of permanency in road-bed and ballast are attained.

Many will agree, probably, that there is good ground for concluding that though the longitudinal is not likely to supplant the cross-tie, it may, before long, be used to supplement it in special cases, such as, for instance, on lines crowded to such an extent with heavy high speed traffic that permanence of way comes to represent not merely a theoretical but a direct and obvious money value, worth saving ; also in some tunnels wherein renewals and maintenance are unusually difficult and obstructive ; also on specially built urban lines, in terminal stations, and on approaches where direct economy is found in perfection of service or where disturbance of the road-bed is not permissible. Already we see illustrations of the latter applications in the costly cable road constructions (parenthetically these show us, too, that rigidity of track is conducive to smoothness of motion and quiet).

Mr. Tratman places, I think, rather undue stress on the difficulty and cost of substituting a new system on an established railroad in operation, although they are undoubtedly of sufficient importance to deter experimental efforts in that direction. On the other hand, we see that the aggregate cost, eventually, of the cross-tie track, after incorporating adequately the various improvements which Mr. Tratman enumerates as being necessary to secure efficiency ; will be brought up to such a figure that the financial argument against introducing the longitudinal features will be greatly weakened, especially if by that time the economy of the additional expenditures shall have been practically proved by increased service.

Whether a track can be evolved, all parts of which can be kept permanently in place and in full efficiency, excepting the top member that wears directly under the wheel, and wherein even the wearing part is compacted and not ruptured by the stresses, is still almost as much as ever a matter of conjecture; but it is at least safe to say that, considered with reference to the present and growing conditions of railroad service, such a theoretically perfect structure would not be either a longitudinal or a cross-tie track, but would be both; that is, it must combine in one all the advantages of each.

E. R. Tratman, Assoc. Am. Soc. C. E.—In reply to Mr. Crowell's discussion, it may be said that the paper refers more especially to the ordinary open track under ordinary conditions, than to exceptional conditions or circumstances, but at the same time states that there is a possibility of the introduction of the longitudinal system at certain localities and under such exceptional conditions, as at bridges, etc. In this connection it may be of interest to note that the tracks in the new Union Station (Louisville and Nashville Railroad), at Louisville, Ky., consist of 80-pound rails laid on continuous iron plates upon longitudinal timbers 12 x 12 inches in section, which rest on wooden cross-ties. The spikes pass through the iron plates. The ballast filling is brought up level with the tops of the longitudinals. Mr. Crowell's suggested combination of longitudinals and cross-ties, presumably similar to the tracks just mentioned, is a development of the old arrangement very generally used for street railways. Such a construction for a railway, however, would require a very large amount of timber. Where heavy steel ties have been used under the joints of steel longitudinals on foreign railways, it has in some cases been found necessary to replace them with lighter connections. It is possible, however, that a complete track on this combination system might be designed, built entirely of steel and having efficient fastenings, which would present advantages of such force as to lead to its trial on busy sections of track with heavy and constant traffic. It would be an expensive construction, but if successful might lead to a considerable reduction in maintenance and repairs.

What the track of the future will be it is not easy or safe to predict, but the improvement of railway track, either radically or in detail, is a problem occupying many minds. The author recognizes the advantages of the longitudinal system, as has already been shown; but its general introduction on the ordinary open track does not seem probable, and under such ordinary conditions of track and traffic it is likely that the present system of track, with cross-ties, can be very much improved at less cost than would be involved in the introduction of a new system.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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515.
(Vol. XXV.—December, 1891.)

FREE RAILWAY CONSTRUCTION vs. GOVERNMENT CONTROLLED AND OWNED RAILWAYS.

By E. BATES DORSEY, M. Am. Soc. C. E.

WITH DISCUSSION.

Under the head of Free Railway Construction will be included all railroads that have been or can be built by individuals or companies, without special acts of Government, whether national or State. Under the head of Government-Owned Railways will be included those owned and operated by the Government; and under the head of Government-Controlled Railways will be included all those that cannot be built without a special act of Government, as is the case with all English railways; in these the English Government has no direct or indirect ownership whatever, but owing to the great difficulty and cost of getting from Parliament special acts to build new lines of railways, the present English roads have a virtual monopoly of the land transportation of the United Kingdom. It is needless to say that the roads make the most of this exclusive right, by charging freight rates that to Americans appear excessive, half of which would fill the hearts of our Railroad Managers with joy.

All that can be said in favor of Government ownership or control of

railways is, that it can regulate the freight charges and prevent excessive rates that would interfere with the industries of the country. Practically it has no such effect. The result of the writer's experience and observation is that the highest railway freight rates in the world are in England and in her colonies, where the railways are either owned or controlled by the Government; and the lowest rates are those in the United States, where there is free railway construction, and consequently great competition, which has obliged our managers to build and operate their railroads upon the most economical plans, and to adopt all new inventions that will reduce the cost of construction or operating expenses. This has made it possible for them to transport freight over inferiorly constructed roads at less than half the English cost.

The following table shows some of the current freight rates in England, South Africa, and the United States:

£1 = $4 80.	Miles of Haul.	Freight Charge Per Ton Per Mile.		
		England.	South Africa.	United States.
		Cents.	Cents.	Cents.
Birmingham to London, hardware	113	*a* 5.85	.....	.....
Birmingham to Manchester, hardware	90	*a* 5.73	.....	.....
Stoke-on-Trent to Liverpool, pig iron	55	*a* 2.32	.....	.....
Birmingham (U. S.) to Pensacola, pig iron	280	.....	.....	*b* .50
Birmingham (U. S.) to Pensacola, coal	280	.....	.....	*b* .50
Merthyr (Wales) to London, coal	177	*a* .995	.....	.....
London to Oldham, cotton	188	*a* 2.56	.....	.....
Liverpool to Oldham, cotton	43	*a* 5.10	.....	.....
Cape Town to Kimberly, grain (domestic)	647	.....	*b* 3.	.....
Cape Town to Kimberly, iron	647	.....	*b* 7.	.....
Port Elizabeth to Kimberly, grain (domestic)	485	.....	*b* 3.	.....
Port Elizabeth to Kimberly, grain (imported)	485	.....	*b* 6.	.....
Cape Town to Kimberly, grain (imported)	647	.....	*b* 6.	.....
Port Elizabeth to Kimberly, iron	485	.....	*b* 7.	.....
Durban to Biggarsberg—Natal, grain	231	.....	*a* 8.	.....
Durban to Biggarsberg—Natal, iron	231	.....	*a* 9.25	.....
Newcastle to Durban—Natal, coal	270	.....	*a* 1.50	.....
Average charge for all freight on Pennsylvania Railroad Division. Average haul	135	.....	.....	*b* .626

Note.— *a* = ton of 2 240 pounds.

b = ton of 2 000 pounds.

The railway from Durban has many grades of one in thirty, and one summit of 5 155 feet above its initial point. The first part of the road is very badly located for cheap working. Coal on the railways from Port Elizabeth and Cape Town costs from $8 to $12 per ton. All the railways in South Africa are owned and operated by the Colonial

governments; they are very well constructed and maintained, much better than the average of our American roads.

In a former paper read by the author before the Society, and on page 112 of his book "English and American Railroads Compared," the author has compared the London and North Western Railway of England to the Pennsylvania Railway Division of the United States. The comparison is here continued down to the year 1889.

Average cost of transporting one ton or one passenger one mile. £1 = $4 80.	London and North Western.		Pennsylvania Railroad Division.	
	1884.	1889.	1884.	1889.
	Cents.	Cents.	Cents.	Cents.
Maintenance of way	.209	.208	.103	.089
Repairs, renewals of locomotives	.082	.085	.044	.034
Total cost of motive power	.271	.286	.148	.130
Total operating expenses	1.130	1.208	.530	.491

This shows that in the last five years the total operating expenses of the London and North Western has increased 7 per cent., while that of the Pennsylvania has decreased the same percentage.

In comparing American railways with those of Europe, the usual mode of comparison between the cost of train miles is misleading and erroneous, owing to the great difference in the train load of paying freight; which is only 59 tons on the Great Northern of England. The general average of all the railways of the United Kingdom is 71 tons—or less than half of the average paying load of the American trains. It is like comparing the cost of a donkey cart with that of a six-horse wagon.

Much has been said of the great advantage the American railways have over the English in their long freight haul; this argument is entirely false, as the average receipts per ton on the English roads are about the same as those on the American, the only difference being that the American hauls the freight more than three times the distance for about the same money. For example, the average haul of the Pennsylvania Railroad Division is 135 miles, with average receipts per ton of eighty-five cents; on the London and North Western Railway of England, the average haul is about 35 miles, and the average receipt per ton eighty-four cents. The author can see no advantage of the long haul in this case.

The severe competition between railways in the United States is undoubtedly very bad for the shareholders, but it has been the making of the country. Suppose we estimate the sphere of beneficial influence to extend 12 miles each side of the line; this gives 15 360 acres of land for each mile of road; estimate this at $20 per acre, and it makes the total valuation of the land benefited by the railway $307 200, while the cost of the mile of railway is, say, $30 000, or 10 per cent. of the value of the land benefited. The appreciation in the value of the land is very much larger than the cost of the railway, and the investment would pay even if the entire cost of the railway was lost.

The leading English railways pay the following dividends on their ordinary shares.

	Per Cent.	
	1888.	1889.
London and North Western	7⅜	7⅜
Midland	6	6
Great Northern	4⅞	4⅞
Great Western	6¼	6¼
London and South Western	6	6
Taff Vale	5	5

This shows that the English railways pay very well when their great cost is considered, but the money is undoubtedly earned at the expense of home industries. This is seen very forcibly in the depreciation of farming lands during the last twenty-five years—since the wheat of our Western States has come into direct competition with the English farmer; notwithstanding that the first has to pay in many cases over 1 500 miles of railway freight and 3 500 miles of freight by sea, while the last has only to pay railway freight for less than 300 miles. In spite of these great disadvantages, the American farmer competes so successfully with the English, that he has depreciated very largely the value of English farms.

If the English freight rates were in force in this district (Eastern Tennessee), the iron and coal interest surrounding us, that is being so extensively worked under our low railway rates, could not exist; and in place of the busy mines and furnaces there would be only a few farmers raising enough food for their own support, shipping, perhaps, a little cotton, for nothing else could stand the heavy railway freight rates to the seaports.

In English South Africa all the railways are owned by the Government, and the evil effects of the state ownership can be readily seen. In order to get the necessary votes in the Assembly to pass a bill authorizing the construction of meritorious lines, it has been necessary to build other lines that were not required and will not pay. To secure the required votes they must make an Omnibus bill, including the different railways proposed; following there in their railway bills the same practice that we do in our river and harbor bills.

As an example of this, the Cape Colony Government railways of South Africa, operating 1 599 miles, which cost £14 282 766, earned in 1889 a net profit equal to a dividend of 5¾ per cent. on the total cost. The road from Port Elizabeth to Kimberley, a part of the system, 485 miles long, costing £4 300 000, earned in the same year 13¾ per cent. on its total cost. This branch transported large quantities of mining machinery and goods to the interior for the diamond and gold mines. This branch shows the remarkable receipt of over $17 for each ton of freight handled, which the author believes to be unequaled on any other road.

As the Government and Railway officers wish to make a good financial showing, they are obliged to charge high freight rates on the good lines, to make up for the loss on the poor ones. This has caused such high freight rates as to be nearly prohibitory to farming except near the sea-coast. Diamond and gold mining, sheep and ostrich farming are the only industries developed in the interior that can stand the high freight charges. The Government will not grant permission to construct private lines, for fear of competing with and injuring the public lines.

Many reasons such as republican government, free government, free religion, rich soil, etc., have been given as the cause of the great prosperity of the United States; but the real cause has been the law allowing free railway construction. This has given the country much greater railway facilities than it could have had otherwise, and the sharp competition has given such low freight rates, that the interior of the United States, notwithstanding its thousand miles of railway freight, can compete successfully in Europe in its food products.

On the English railways, which have cost from four to six times more to construct than the American roads, the cost for transporting freight is more than double the American cost, whereas it should be much less, or why make the increased outlay and expense for superior construction?

The author thinks that the great difference in favor of American rail-

ways in the cost of operating expenses is owing principally to the following reasons:

First.—The trains on the American railways carry much larger loads than those on the English roads.

Second.—The universal use on the American railways of rolling stock with bogie-trucks which run with much less friction and wear and tear than the English rolling stock, with its long, rigid wheel base.

Third.—The general use on the American railways of freight cars or wagons carrying a greater percentage of paying load to the dead weight than those used on the English railways.

Fourth.—The lower speed at which the American goods or freight trains run.

Fifth.—The use on the American railways of heavier locomotives, which haul heavier loads than can be done by the lighter ones used on the English railways.

Sixth.—The use of locomotives on the American railways with outside cylinders and connections, which can be more cheaply and easily repaired than those with inside cylinders used on the English railways.

Seventh.—The use on the American railways of collecting and distributing freight trains which load and unload at stations freight in less than car load quantities, thus avoiding leaving cars only partially loaded.

The best authority gives the average load of the English freight car at from $1\frac{1}{2}$ to $2\frac{1}{2}$ tons, while the weight of the wagon will average over 5 tons—making the tare more than double the paying load. As soon as the English Railway managers are convinced, and will act upon this conviction, that it costs as much to haul 1 ton of tare or dead weight 1 mile as it does 1 ton of paying load, they will then make a very large saving in the cost of transportation, by reducing the present excessive tare.

If there was free railway construction in the United Kingdom, with new roads built and run also, according to modern railway practice, these would by competition force the old roads to improve their rolling stock, and consequently reduce largely their operating expenses, thus enabling them to make large reductions in their freight charges, without interfering with their dividends.

Those who are working for State interference, control, or ownership of railways, in order to reduce the freight charges, should travel and see what is being done in other countries, where the railroads are more or

less controlled by the Government; they would return convinced that railway facilities and freight charges in the United States, under our free railway construction laws, are by far the best and the lowest in the world. They should look into the financial condition of the offending road, when, if they can show large or excessive profits, it would be very easy to obtain capital to build competing lines—promoters and contractors being constantly on the lookout for such opportunities. The most dissatisfied Granger cannot say that our railways are now paying unreasonably large dividends.

The author, after long and close observation in other countries, is of the firm conviction that the best way to reduce freight rates is to have free railway construction and operation and its consequent competition; as it will give cheaper rates and greater facilities than can be had under Government-controlled or owned railways and Government management.

DISCUSSION.

Robert Moore, M. Am. Soc. C. E.—I think this paper might very well be circulated as a tract among our Western legislators. It shows very completely that the people at large have no reason to ask for State ownership of railroads. But the showing for the railroads themselves is not so good, and it is a question whether the State might not properly interfere to check somewhat the unlimited construction of railroads which prevails in our country now. There have been under our present system of free railroad construction cases which might almost be called blackmail, of which the benefit either to the community or to the owners of the road is extremely doubtful. So that if a method can be found by which the construction of unnecessary railroads can be avoided, I think that the whole community will be the gainer. It is worth considering whether the partial return to the English practice which is now in vogue in the State of Massachusetts is not advisable. By this system a charter under the general statute is granted only when the incorporators can prove to the Railroad Commission that the road which they seek to construct will be a public benefit, and that they are able to build it. Failing in this, they must go to the Legislature for a special act of incorporation.

Something like this would be a very wholesome check upon overconstruction, and may well be commended for adoption in the older States at least.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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(Vol. XXV.—December, 1891.)

DISCUSSION ON PAPER No. 493.

By J. C. TRAUTWINE, Jr., EDWIN R. KELLER, F. COLLINGWOOD, G. W. PLYMPTON.

J. C. TRAUTWINE, Assoc. Am. Soc. C. E.—In my discussion of Mr. Plympton's paper on the strength of plate glass, I remarked that he left us without the connecting link between his experiment on bars and their application to plates, and his reply hardly answers the question. I found that Grashof gives formulas for the strength of plates with their edges confined; and on application to Edwin R. Keller, of the University of Pennsylvania, he furnished the following discussion:

EDWIN R. KELLER, M. Am. Soc. C. E.—The experiments of Professor Plympton on the transverse strength of Dutch plate glass, were not performed under the conditions to which glass plates are subjected in actual practice; yet we may, by the following method, which is similar to that adopted by Weisbach in the discussion of flat boiler sheets, formulate the results in such a way that they will obtain approximately for the case of a rectangular plate fixed in water-tight fastenings and uniformly loaded.

Consider the plate to be made up of a number of strips such as those used in the tests, and suppose them to be placed with the length (a) horizontal. Suppose a strip of length a and width unity to carry per

linear inch the portion w_1 of the unit pressure w. Then if R is the unit stress in the most strained fiber and d the thickness, we have:

$$\frac{1}{12} a^2 w_1 = \frac{R d^2}{6}$$

and

$$d = a \sqrt{\frac{w_1}{2 R}}.$$

The max. deflection occurs at the center and is given by

$$\delta_1 = \frac{1}{384} \frac{12 w_1 a^4}{d^3 E}.$$

Similarly if we consider the plate made up of a number of strips of length b, placed vertically and carrying the portion w_2 of the pressure w_1 on each linear inch, we obtain:

$$d = b \sqrt{\frac{w_2}{2 R}} \quad \text{and} \quad \delta_2 = \frac{1}{384} \frac{12 w_2 b^4}{d^3 E}.$$

Now, evidently, the deflections δ_1 and δ_2 must be equal, and therefore,

$$\frac{\delta_1}{\delta_2} = 1 \quad \frac{w_1 a^4}{w_2 b^4} \quad \text{and} \quad w_2 = \left(\frac{a}{b}\right)^4 w_1$$

$$w = w_1 + w_2 = \left(1 + \frac{a^4}{b^4}\right) w_1$$

$$w_1 = \frac{w b^4}{a^4 + b^4} \qquad w_2 = \frac{w a^4}{a^4 + b^4}$$

$$\therefore \quad d = a \sqrt{\frac{w b^4}{2 R (a^4 + b^4)}} = b \sqrt{\frac{w a^4}{2 R (a^4 + b^4)}}$$

And, in general,

$$d = a^2 b \sqrt{\frac{w}{2 R (a^4 + b^4)}}$$ where a is the larger side of the rectangle.

Considering now the experimental results, we have, for a beam supported at both ends and loaded in the center:

$$\frac{1}{4} W l = \frac{R b d^2}{6} \text{ or } R = \frac{3}{2} \frac{W l}{b d^2}$$

and, substituting the values obtained in the time tests, we obtain:

Experiment.	R.
No. 4	3 609
8	3 955
9	4 500
Mean	4 020

Using this value in the formula, we obtain, for the proper thickness:

$$d = \frac{a^2\ b}{89.7} \sqrt{\frac{w}{a^4 + b^4}}$$

a being the larger and b the smaller dimension of the pane in inches, and w the wind pressure in pounds per square inch.

George W. Plympton, M. Am. Soc. C. E.—I am inclined to the opinion that Grashof's method as given by Lanza would afford a more satisfactory approximation than that given by Mr. Keller. The significance of w_1 and w_2 needs to be more clearly stated by Mr. Keller. As the text reads, either of them represents the pressure on a unit area.

F. Collingwood.—There is one suggestion which occurs to me, viz., whether plate glass of any size could be considered as fixed at the edges. The hold in any case is usually quite narrow, and I very much doubt whether the glass is strictly in the condition called "fixed."

Mr. Keller.—It seems to me that the significance of the quantities w, w_1 and w_2 is stated as clearly as may be; they are not equal to each other, but w_1 is that portion of w which is carried by a longitudinal strip and w_2 is that portion of w which is carried by a transverse strip, so that $w = w_1 + w_2$, as given in the demonstration.

Referring to Mr. Collingwood's suggestion that a glass plate "is not strictly in the condition called fixed," because the hold is narrow, I would say that since a beam is fixed when the tangent to the elastic curve at the supports is horizontal, it appears to me that if the hold is sufficiently rigid the plate will be fixed, no matter how narrow it is. Rupture is rarely caused in a glass plate by the stress which a given load would theoretically produce, but generally by the inequality in the support of the ends. It is therefore impossible to make any but the roughest calculations. The formula is given only as a rough approximation and as such must be used with an ample factor of safety.

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(Vol. XXV.—December, 1891.)

EXPERIMENTAL DETERMINATION OF THE ROLLING FRICTION IN OPERATING THE DRAW OF THE THAMES RIVER BRIDGE, TOGETHER WITH METHOD FOR DETERMINING POWER TO OPERATE DRAW-BRIDGES.

By ALFRED P. BOLLER, Jr., and H. J. SCHUMACHER.

WITH DISCUSSION.

The following paper on the frictional resistance of draw-bridges is submitted to supplement the very meager data on the subject. The only recorded experiments the writers have been able to discover are those published in the *Transactions* Am. Soc. C. E. some twenty years ago, in a paper by C. Shaler Smith on the general subject of draw-bridges. These experiments were made by simple dynamometer measurements, and took no account of the moment of inertia of the structures experimented upon. Further, they were made on comparatively small bridges, and on a quality of workmanship far below what is now within shop possibilities. A careful experimental record, with the deductions therefrom, upon a large modern draw-bridge, was thought to be of sufficent value to be made a matter of permanent record, and yielding useful working data in a field in which little work has been done.

The Thames River Bridge at New London, Conn., was selected, as it represents the greatest achievement thus far in draw-bridge building. It is 500 feet long, weighs 2 400 000 pounds, and is the largest double-track draw in the world. The management of the N. Y. P. & B. R. R. gave all assistance necessary, otherwise the experiments could not have been undertaken. The bridge was opened in the fall of 1889, and had been in successful operation a year and a half when the observations were made from which the results were deduced. The greatest skill and care were exercised in making all the members of the bridge, and particularly in making and setting up the turn-table and track, so as to reduce to a minimum the work of operating due to imperfect workmanship and irregularities.

The following is a description of the turn-table taken from the report by the Chief Engineer:

"The whole weight of the draw is delivered to a rim-bearing turn-table at four points, the supporting cross girders being arranged so as to deliver the weight in exact proportion to eight equidistant points on the circular drum, which, being 32 feet in diameter, divides into $12\frac{1}{2}$ feet segments which measure the extent of the distribution the drum is called upon, to afford a uniform bearing on all the wheels under the influence of each segment. The whole weight rests upon fifty-eight cast-steel 20-inch wheels, with 10-inch face of metal as hard as could be faced in the lathe, weighing 800 pounds each. The estimated weight borne by these wheels is 1 300 tons, or $22\frac{4}{10}$ tons per wheel, being 4 400 pounds per lineal inch of face. The wheel treads are rolled steel plates bent to the curve, and faced to a true cone and bearing.

"Extraordinary care was exercised to make the lower tread absolutely level, having all bearings and surfaces as true as machines could make them, and no expense was spared to make the drum rigid and stiff, as the price of a smooth and easy working table. The live ring coupling the wheels is perfectly flexible, being composed of separate band pieces between each pair of wheels. The rack is of cast-iron, with a face of 10 inches, teeth $3\frac{1}{2}$ pitch, and a pitch circle of $34\frac{1}{3}$ feet in diameter, into which gears two shrouded pinions with thirteen teeth and 15-inch pitch circle, having vertical shafts $5\frac{1}{2}$ inches diameter, working in heavy and strongly braced sleeves attached to the drum. The table is provided with both hand and steam power, the former through capstan bars, three for each shaft, engaging into capstan heads attached to the multiplying gear driving the vertical shaft.

"The steam power located within the drum consists of a pair of oscillating cylinders 10 inches in diameter with a 7-inch stroke, on a compact frame setting at an angle of 120 degrees, and working upon one crank shaft; running at a maximum velocity of 200 revolutions (average about 170), applying the power through two Frisbie friction clutches, the one driving the unlocking shaft, the other the turning shaft. The turning shaft, of $4\frac{7}{16}$ inches diameter, runs at a speed of nine turns per minute, gearing direct through a pair of beveled gears into the vertical drum shafts."

Method Pursued in Determining Friction.—Of the work done by the engine in operating a draw-bridge, part gives acceleration to the mass and produces velocity of rotation in the draw-span, the remainder being absorbed in friction. It was found impracticable to "indicate" the engines on account of the oscillating type, also on account of the variation of the velocity and the short time of runs, the engines being in operation less than two minutes. Therefore another method was pursued, as follows: The engines were run until the bridge attained a certain velocity; they were then thrown off and the bridge allowed to come to rest. The force producing this retardation must equal the force producing the work of friction. All runs being made during absolutely calm weather, no consideration of wind effect was necessary.

Description of Apparatus for Determining Velocities.—A continuous record of the velocity of the bridge was kept upon a strip of paper, which being drawn through by rolls moved by the main driving shaft of the bridge, moved proportionately to the velocity of rotation of the bridge. Upon this strip of paper a mark was made by a pencil point attached to the armature of an electro-magnet, the magnet being charged every two seconds by an electric contact made by a torsion pendulum; equal intervals of time were consequently marked upon the paper. The paper used was a heavy quality of manilla cut in strips $3\frac{1}{2}$ inches wide and about 30 feet long. A number of records of swings could be taken upon these strips, as both sides were used. A strip was rolled up and placed on a spindle at the end of the guide box. This was simply a box closed upon three sides and open at the ends, through which the paper was fed. A block of wood, pressed upon by an adjustable spring, pressed the paper against the bottom of the box, producing the desired tension. From this box the paper passed to the marking apparatus, the paper passing through guides and over a marking plate as it passed under the pencil point. The paper was then led to the feed rolls. These consist of an arbor, accurately lined and firmly fastened, upon which turns a well-fitted sleeve, this sleeve being turned by a pulley fastened to it, which is driven by a belt from the main driving shaft. A loose roll, pressed down upon the sleeve by means of springs and being prevented from turning about the spindle, it, together with the sleeve, drew the paper, which was led along between them, at a rate proportional to the turning of the bridge.

A torsion pendulum was used, as the length of a simple gravity pendulum which would give the required interval of time, was too great to allow of its being swung on the bridge; and as no suitable clock could be procured, and a torsion pendulum would run quite accurately for the short time (three minutes) required for a swing. The pendulum was located upon the stone pier under the radial spider rods. This position was so chosen because the vibrations of the bridge were less and because the motion of the bridge could not influence the swinging of the pendulum in any manner. The pendulum was constructed of two well seasoned pieces of pine, fastened together in the shape of an inverted T and suspended by a piece of tempered steel wire. Upon the cross-piece and equidistant from the center were placed two equal weights, and by varying their distance from the center of oscillation the time of vibration might be adjusted. The weights were adjusted by timing the pendulum by a watch and counting the clicks of the magnet for six minutes; in that time the pendulum would not vary a tenth part of a second. The whole was incased in a box to protect it from draughts, etc., a glass being placed in the top so that the operation of the pendulum might be observed. The suspension wire passed through a close-fitting hole in a sheet of metal at the top of the box, and was fastened to a horizontal piece of metal, a movement of which, in a horizontal plane, would give a purely torsional movement to the wire. The electric contact was made by means of a wire leading from the torsion wire and a drop of mercury in an adjustable block of wood. The mercury was placed in the center of oscillation and the contact wire passed through it twice during each swing.

*Method of Calculating the Moment of Inertia.**—The moment of inertia, I, of the draw was calculated by breaking the whole into parts as follows, the total I being the sum of these parts:

First.—Inertia of the truss, the weights of the drum, etc., being considered as part of the truss.

(*a.*) About an axis traversing the center of gravity of the truss parallel to the axis of rotation. This was found by multiplying the weight of the truss, reduced to joint loads, by the squares of their distances from the axis of the truss.

* All the weights used in the calculation of the moment of inertia are the actual shipping weights of the members, as weighed at the shop.

(*b.*) The total weight of the truss into the square of its distance from axis of rotation.

Second.—Inertia of floor beams.

(*a.*) About an axis traversing the center of gravity parallel to the axis of rotation.

(*b.*) The weight of each floor beam into the square of its distance from the axis of rotation.

Third.—Inertia of stringers, rails, and guard-rails.

(*a.*) About an axis traversing the center of gravity parallel to the axis of rotation.

(*b.*) The weight of these pieces into the square of their perpendicular distance from the axis of rotation.

Fourth.—Inertia of timber, ties, etc.

Considered as a flat rectangular strip of uniform weight per cubic foot about the axis of rotation.

CALCULATION OF THE MOMENT OF INERTIA. 1 *a* and 2 *b*.

Truss No.	Distance.	Distance.^{2}	Truss.		Floor Beams.	
			Weight.	Product.	Weight.	Product.
0	248.74	61 851.69	17 700	1 094 775 090	13 400	828 812 780
1	224.91	50 580.01	26 410	1 335 817 800	8 830	446 621 400
2	201 07	40 441.21	25 660	1 037 721 192	8 830	357 095 796
3	177.24	31 399.84	29 660	931 318 068	8 830	277 260 234
4	153.41	23 381.56	30 160	705 189 056	8 830	206 459 528
5	129.57	16 796.16	33 660	565 360 092	8 830	148 310 446
6	105.74	11 172.49	38 160	426 342 600	8 830	98 653 175
7	181.91	6 707.61	34 660	232 485 416	8 830	59 228 108
8	58.07	3 375.61	39 660	133 876 296	9 830	29 806 548
9	34.24	1 169.64	38 660	45 216 736	8 830	10 327 568
10	10.41	108.16	92 660	10 025 812	8 830	955 406
	Sum		407 050	6 518 128 158		2 463 530 989

This is for only half of one truss, and the first total product should therefore be multiplied by four for the total. It is also for half of the span for the floor beams, and this product should be multiplied by two for the total. The trusses are 14.16 feet from the axis of rotation.

2 *a*. For simplicity's sake consider each floor beam to consist of a rectangle 27 × 4 feet × 2 inches; then since $I = \frac{1}{3} W (b^2 + c^2)$, in which $W = 8\,830$, $2b = 27$ feet and $2c = 2$ inches, $I = \frac{1}{3} \times 8\,830 \times 182\,257 = 536\,440$. There are twenty-two floor beams.

3 *a*. Consider a rectangular strip 500 × 2.5 feet × 2 inches; then since $W = 90\,533$, $2b = 500$ feet, $2c = 2$ inches, $I = \frac{1}{3} \times 90\,533 \times 62\,501.563 = 1\,886\,170\,000$.

There are four stringers, etc., two distant 3 feet and two distant 9 feet from the axis of rotation.

4. For the floor beams $W = 206\,667$ and the dimensions are 500 × 18 feet × 9 inches; hence $I = \frac{1}{3} \times 206\,667 \times 62\,824 = 4\,102\,560\,000$.

Sum of I.

1 *a*.	$6\,518\,128\,158 \times 4 =$	26 072 512 632
1 *b*.	$407\,050 \times (14.16)^2 \times 4 =$	326 454 100
2 *b*.	$2\,463\,530\,989 \times 2 =$	4 927 061 978
2 *a*.	$536\,440 \times 22 =$	11 801 680
3 *a*.	$1\,886\,100\,000 \times 4 =$	7 554 400 000
3 *b*.	$90\,533 \times 3^2 \times 2 =$	1 629 594
3 *b*.	$90\,533 \times 9^2 \times 2 =$	14 666 364
4	.....	4 102 560 000
		43 011 086 348

Weights.

Trusses, drum, etc., 4 × 407 050.....	1 628 200
Floor beams, 8 830 × 20 + 13 400 × 2.....	203 400
Stringers, rails, etc., 90 533 × 4.....	362 132
Timber.....	206 667
	2 400 399

$$\rho^2 = \frac{I}{W} = \frac{43\,011\,086\,348}{2\,400\,000} = 17\,921.286$$

$$\rho = 133.87 \text{ feet.}$$

Determination of the Acceleration.—Having found ρ the radius of gyration, it was necessary to find the negative acceleration at ρ distance from the axis of rotation, in order to determine the force producing the negative acceleration, which is the frictional resistance. From the proportion of the paper rolls, etc., it was found that 1 inch of paper was equivalent to 3.1218 feet at ρ. The acceleration in inches was found from the formula $f = \frac{2s}{t^2}$, in which $f =$ the acceleration, s the distance in inches of paper corresponding to that from the throwing off of the engine to the point of cessation of motion of the bridge, and t the number of seconds indicated on the strip between these points. This was multiplied by 3.1218 to give the acceleration at ρ in feet. This was done for each card, and the average acceleration taken. As the friction is assumed

to be constant, the acceleration is consequently assumed to be constant. The table shows, however, variations which apparently following no law. These include slight variations in friction for different portions of the track, unobservable wind pressures, imperfections of apparatus and errors of observation. The mean of so large a number of observations must, however, give a value probably nearly correct.

TABLE OF ACCELERATIONS FROM CARDS REDUCED TO ρ DISTANCE FROM CENTER.

Card No.	Time $= t$ in seconds.	(Time) ^{2}	Length card in inches $= s$.	Acceleration in feet. $f = \frac{2s}{t^2} \times 3.1218$
5.....	56	3 136	8 438	.01680
6.....	48	2 304	6 438	.01745
7.....	80	6 400	15 375	.01501
8.....	82	6 724	17 750	.01555
9.....	86	7 396	17 250	.01456
10.....	100	10 000	23 375	.01460
11.....	78	6 084	16 188	.01662
12.....	48	2 304	5 688	.01541
14.....	118	13 924	29 469	.01320
15.....	116	13 456	36 000	.01666
16.....	102	10 404	24 219	.01454
17.....	144	20 736	45 594	.01373
18.....	134	17 956	40 188	.01397
19.....	90	8 100	17 547	.01356
20.....	118	13 924	28 422	.01285
21.....	110	12 100	23 875	.01233
22.....	142	20 164	44 531	.01379
24.....	74	5 476	14 563	.01663
25.....	44	1 936	5 844	.01886
26.....	80	6 400	15 719	.01533
27.....	82	6 724	16 094	.01493
28.....	92	8 464	19 906	.01469
29.....	124	15 376	32 313	.01312
	Sum.....	.....	.....	.34419
$\div 23 =$	Average.....	.....	.....	.01496

Frictional Resistances at Center of Rollers—

Let R_1 = radius to center of rollers = 16 feet.

ρ = radius of gyration = 133.87 feet.

F_1 = force at center of rollers due to f,

Let F = force at extremity of ρ due to f.

f = acceleration.

W = weight of bridge.

ϕ = total co-efficient of friction.

$$\text{Then } F_1 = F\frac{\rho}{R_1} = Mf\frac{\rho}{R_1} = \frac{W}{g}f\frac{\rho}{R_1}$$

$$F_1 = \frac{2\,400\,000}{32.2} \times .015 \times \frac{133.87}{16} = 9354$$

$$\phi = \frac{F_1}{W} = \frac{9354}{2\,400\,000} = .00389$$

This total amount of friction is due to:

1. Collar friction of bearings on live ring.
2. Overhauling shafting and gearing.
3. Rolling friction.

Collar Friction of Bearing on the Live Ring.—The friction due to the pressing of the roll against the live ring cannot be determined experimentally. The theoretical assumption made, however, which cannot depart far from the truth, was that the outward horizontal pressure will be the component of the weight of the bridge, pressing vertically, due to the obliquity of the rolls. This would be absolutely true if the wheels and track were absolutely true and frictionless. Of course this cannot be realized in shop practice. If the apex of the conical rollers falls beyond the center of rotation, the roll will tend to move outward when the bridge turns, increasing the outward pressure. If it falls short of the center, conversely, it will tend to roll in, decreasing the pressure. These will tend to neutralize each other, and, together with the jarring and vibrating of the bridge and the rolling action of the rolls, may cause the full theoretical horizontal component. Assume that this is so, then let $H =$ the horizontal component, $W =$ weight of bridge, $H = 2\ W$ tang. α, and we have the total outward strain $H = 2\,400\,000 \times \frac{2 \times 10}{16 \times 12} = 250\,000.$

For collar friction we have (see Rankine, A. M., page 251). Lever arm of friction $= \frac{2}{3} \frac{r^3 - r^3_1}{r^2 - r^2_1} = 1.7$ inches.

Assume $\phi_2 = .07$ and let $W_1 =$ work done by collar friction of live ring bearing for 90 degrees swing, then

$$W_1 = 2\ \pi \times \frac{1.7}{12} \times .07 \times 250\,000 \times \frac{\pi \times 12 \times 32}{4 \times \pi \times 20} = 74\,760 \text{ foot-pounds.}$$

Work Lost by Gearing.—From "Konstructeur" — F. Reuleaux.

$p_r = \pi\ \phi_3 \left(\frac{1}{Z} + \frac{1}{Z_1}\right) \frac{E}{2}.$

$\phi_3 =$ co-efficient of friction $= .15.$

$E =$ length of arc of contact $=$ $1.6 \times$ pitch (Rankine, M. M.).

$p_r =$ loss of work.

Z and $Z_1 =$ number of teeth.

Then for bevel gears, $E = 2.51$, and

$$p_r = 3.14 \times .15 \left(\frac{1}{17} + \frac{1}{45}\right) \frac{2.51}{2} = .048.$$

For pinion and rack, $E = 5.4$, and

$$P_r = 3.14 \times .15 \left(\frac{1}{360} + \frac{1}{13}\right) \frac{5.4}{2} = .10.$$

Work Done in Overhauling Horizontal Shaft.—Weight of shaft $= 2\,500$ pounds.

For 90 degrees swing the shaft turns 18.33 times.

W_2 = the work done by the horizontal shaft for 90 degrees swing.

The distance the shaft moves through = 4.5 inches $\times \pi \times$ 18.33 = 259 inches. Assume $\phi = .05$, then

$$W_2 = .05 \times \frac{259}{12} \times 2\,500 = 2\,698 \text{ foot-pounds.}$$

Friction in the Collar of the Vertical Shaft.—The lever arm of the friction = 6.4 inches, $\phi_2 = .07$. Weight of shaft = 1 000.

$W^1{}_2$ = work absorbed in friction for 90 degrees swing of the shaft turning 6.92 times.

$$W^1{}_2 = 6.92 \times \frac{2\,\pi \times 6.4}{12} \times 1\,000 \times .07 = 1\,622 \text{ foot-pounds.}$$

Total Work Done by Overhauling Shafting, etc.—

$$\frac{\frac{2\,698}{(1 - .05)} + 1\,622}{(1 - .10)} = 5\,053 \text{ foot-pounds for 90 degrees swing.}$$

Relative Work Done by Friction.—This work for 90 degrees swing of the bridge is:

	Foot-pounds.
For collar friction of the live ring bearings.....	74 760
" overhauling gearing, etc.....	5 053
	79 813

The total work of friction $= 9\,354 \times \frac{1}{4} \times \pi \times 32$

$= 234\,972$ foot-pounds.

$79\,813$ " "

Work of rolling friction = $155\,159$ " "

The percentage of friction due to—

1. Collar friction of live ring bearing is ..... $= \frac{74\,760}{234\,972} = 31.8$ per cent.
2. Overhauling, shafting, etc.... $= \frac{5\,053}{234\,972} = 2.1$ "
3. Rolling friction ..... $= \frac{155\,159}{234\,972} = 66.1$ "

Co-efficient of Rolling Friction.—

ϕ_1 = Co-efficient of rolling friction. w = work.

F_2 = Force required to move W. S = space moved through in 90 degrees of swing.

$$\phi_1 = \frac{F_2}{W} \qquad w = F_2 \times S\text{; hence}$$

$$\phi_1 = \frac{w}{S \times W} = \frac{155\,159}{\frac{1}{4}\pi \times 32 \times 2\,400\,000} = .00257.$$

Another Method for Determination of ϕ.—The determination of the total friction by moving the bridge by the application of a force at the end, was accomplished in the following manner: From the end of the draw, a line was attached leading over a sheave at the end of a strut run out from the fixed span; at the end of this line weights were placed, until the bridge just moved at a uniform velocity. Hence if ϕ = co-efficient of friction; F = weight applied = 530 pounds; F^1 = weight if applied at the track circle = $530 \times \frac{250}{16}$; W = total weight of bridge.

$$\phi = \frac{F^1}{W}; \quad \phi = \frac{\frac{250}{16} \times 530}{2\,400\,000} = .00345$$

Considering the crude manner in which this last experiment was performed, as the uniform velocity was judged only by eye, and as it considered only a small section of the track, the weight, 530 pounds, was the mean of a large number of runs, being quite a variable quantity; it agrees approximately with the first and more accurate method—11 per cent. difference, showing the first figure to be not far from the truth.

PART II.

Method for Determining Power to Operate Draw-bridges.

It is proposed to discuss only that class of draw-bridges in which the whole weight of the structure is supported by a drum resting upon friction rolls, and the center pin only serves to prevent lateral motion. The surfaces of contact of the drum, rolls and supporting track are conical, and have a common vertex in the axis of the drum. The rolls are held in place relatively to the center by spider rods. These rods are fastened to a collar on the center-pin cap, and pass through the rolls. Outside the rolls, and on the rods, are collars or washers, against which the rolls bear. Anchored to the pier, outside the track, is a large gear. Two pinions on vertical shafts, diametrically opposite to each other, mesh with this gear for the purpose of swinging the bridge. The vertical shafts are connected through bevel wheels with a horizontal shaft driven through gearing by the engine.

Resistances.—The resistances to be overcome in swinging a draw are friction, inertia and wind pressure. The frictional resistances are rolling friction, sliding friction between the collars on the spider rods and the rolls, and friction of gearing. The resistance of inertia is the force required to accelerate the motion of the bridge. The resistance of wind pressure is indeterminate, and can only be approximated. If the wind blew perfectly steady, it would have no effect upon the swinging of the bridge, as the pressure on one side of the center would equalize that on the other, whatever the direction of the wind. The wind, however, never blows steadily, but in gusts, and locally, so that for a long span draw-bridge the velocity at one end seldom equals that at the other. When it is desired that the draw should be operated quickly, it is necessary that the engines should be capable of developing sufficient power to swing the draw, when the maximum and minimum velocities of the highest wind under which it is considered safe to operate the draw are affecting the opposite ends at the same time. The effect of wind was very noticeable while conducting the experiments on the Thames River bridge ; as, in high winds, and with the throttle at a constant opening, irregular accelerations and retardations could be noticed in the increase and decrease in the revolutions of the engines.

In determining the resistances it will be most convenient to determine their equivalents at the pitch circle of the large gear or rack, as it is called, since it is at this point that the force to overcome them is applied.

Rolling Friction.

Let R_1 = radius to center of track.
R = radius of rack.
W = weight of bridge.
ϕ_1 = co-efficient of rolling friction.

Let F_r = force at rack required to overcome rolling friction.

Then $F_r = \phi_1 W \frac{R_1}{R}$

Collar Friction.—For the friction of washers and collars at the end of the spider rods.

Let r_1 = interior radius of collar.
r_2 = exterior " " "
r = radius of rolls.
ϕ_2 = co-efficient of friction.

Let W, R, and R_1 be as above.
F_c = force at rack to overcome collar friction.

Then the force with which the rolls are pressed against the collars $= W 2 \frac{r}{R_1}$.

From Rankine, A. M., we have the lever arm of the friction or the radius of the circle at which the total friction may be considered to act:

$$= \frac{2}{3} \times \frac{r_2^3 - r_1^3}{r_2^2 - r_1^2}.$$

The ratio of the force at the rack to the force at the track required to overcome the collar friction $= \frac{R_1}{R}$.

$$\text{Then } F_c = \frac{2}{3} \times \frac{r_2^3 - r_1^3}{r_2^2 - r_1^2} \times \frac{1}{r} \times \frac{R_1}{R} \times \phi_2 W \times 2\frac{r}{R_1}$$

$$= \frac{4}{3} \frac{r_2^3 - r_1^2}{r_2^2 - r_1^2} \frac{\phi_2 W}{R}.$$

Friction of Gearing.—For this we have from Konstructeur—*F.* Reuleaux:

$$p_r = \pi \phi_3 \left(\frac{1}{Z} - \frac{1}{Z_1} \right) ea.$$

Where p_r = percentage of work lost in transmission through gearing.

ϕ_3 = co-efficient of friction. a = a constant depending upon

z and z_1 = number of teeth of gears. the nature of the teeth.

e = arc of contact.

Let $p_1\ p_2\ p_3$ .... be the percentages of work lost in the transmission through the several pairs of gears, w = the work done at the rack and w_1 = the work done by the engine, which is the sum of the work done at the rack and the work lost in transmission through the gearing.

$$\text{Then } w_1 = \frac{W}{(1 - p_1)\ (1 - p_2)\ (1 - p_3)}$$

In subsequent calculations, it will be convenient to consider the gearing frictionless, and increase the actual resistances at the rack in such manner as to make the work of the engine the same as when the actual resistances at the rack and the friction of gearing are considered. Thus, letting

F_1 = the sum of the forces to be overcome at the rack;

F = the force at the rack required to overcome F_1, and assuming the gearing to be frictionless;

Then $F = F_1 \dfrac{1}{(1-p_1)\ (1-p_2)\ (1-p_3)\ \ldots\ldots\ \ldots\ldots\ \ldots\ldots\ \ldots\ldots}$

In this equation the factor $\dfrac{1}{(1-p_1)\ (1-p_2)\ (1-p_3)}$ is a constant for a given draw-bridge. Calling it a, we have $F = a\,F_1$.

Inertia.—To determine the force required to overcome the inertia of the draw, it is necessary to determine its moment of inertia in reference to the axis of rotation. This, by a well known principle of mechanics, will be the sum of the moments of inertia of each part in reference to an axis through its center of gravity, parallel to the axis of rotation of the draw, plus the sum of the mass or weight of each into the square of its distance from the axis of rotation. A practically accurate method is that used in the experiments on the Thames River Draw-bridge. Sufficient accuracy may be obtained by considering the draw to be a rectangular parallelopipedon of uniform density, of the same weight as the draw, and whose length and breadth are the same as those of the draw, and whose depth, or dimension parallel to the axis of rotation is any convenient quantity.

To find the mass which, placed at the radius of the rack, would produce the same moment of inertia:

Let R = radius of rack;
M = mass at R;
I = moment of inertia of draw; then

$$M = \frac{I}{R^2}$$

And letting F_1 = force required to accelerate the draw, or overcome its inertia,
f = the acceleration at the rack; then

$$F_1 = Mf.$$

Wind.—To find effect of wind, let p = the pressure on any member due to the difference of maximum and minimum velocity of the highest wind under which the draw is to be operated, and suppose the wind to blow perpendicular to the length of draw. Then let f = the distance of the center of pressure of any member from the vertical plane through the center of rotation and parallel to the direction of the wind; and let F_w = a force which placed at the rack would produce a moment equal to the moment of the wind pressure. Then, since R = the radius of the rack, we have

$$F_w = \sum_{o}^{n} \frac{f\,p}{R}$$

in which n = the number of members exposed to wind pressure on one end of the draw. With sufficient accuracy we may consider the total wind

pressure as acting at a distance l from the center = one-quarter the length of the draw, when, letting P = the total wind pressure, we have $F_w = \frac{P\,l}{R}$.

Design of Engine.—Since the force required to swing the draw is the sum of the forces required to overcome the various resistances, we have from preceding formula, letting F = this force, $F = a\,(F_r + F_c + F_w)$.

In the second factor of the right member F_r, F_c and F_w are constant for a given bridge, so calling their sum c, we have $F = a\,(c + F_1) = a\,(c + Mf)$.

Acceleration.—To find the acceleration on the assumption that the draw is uniformly accelerated for the first half of the swing, and uniformly retarded for the last half, let t equal the time in seconds in which it is required to swing the bridge, then

$$f = \frac{2S}{\left(\frac{t}{2}\right)^2},$$

in which S = one-eighth the circumference of the rack = one-eighth $(2\,\pi\,R)$.

$$\therefore f = \frac{2\,\pi\,R}{t^2} \text{ and } F = a\,(c + M\frac{2\,\pi\,R}{t^2}).$$

If it is desired to operate the draw by accelerating it until it attains a given velocity which is to be maintained until the brakes are put on and the bridge brought to rest:

Let T = time required for swing;
L = ¼ circumference of rack;
v = velocity at circumference of rack;
l = space on circumference of rack in which draw is to be accelerated;

Then $L = (l + l_1)$ = space in which the draw is to have uniform velocity;
t = time of acceleration;
t_1 = time of uniform velocity;
t_2 = time of retardation;
then $T = t + t_1 + t_2$

l_1 = space on the circumference of the rack in which the draw is to be retarded;

$$\text{but } t = \frac{2l}{v},\ t_1 = \frac{L - (l + l_1)}{v},\ t_2 = \frac{2l_1}{v};\ \therefore T = \frac{L + l + l_1}{v}.$$

In this equation l and l_1 must be assumed, when either v or T may be assumed and the other found. We have

$$f = \frac{v_2}{2\,l};\ \therefore F = a\left(c + M\,\frac{v^2}{2\,l}\right).$$

Dimensions of Cylinders and Multiplication of Force by Gearing.—In the designing of engines for draw-bridges the size of engine may be limited on account of the small space for machinery. In this case the multiplication of force by gearing must be determined.

Let v = greatest linear velocity at rack;
n = greatest number of revolutions per minute of engine;
$\frac{l}{2}$ = length of crank;
R = radius of rack;
m = ratio of length to diameter of cylinder;
F = force at rack, as in previous formulas;
N = number of revolutions of the engine to one of the draw;
p = steam pressure.

The work per second at the engine must equal Fv, and

$$Fv\ 2pl\ \frac{\pi m^2 ln^2}{4\ \ 60}\text{; but } \frac{n}{60} = \frac{v}{2\pi R}\ N\text{; } \therefore\ N = \frac{4RF}{pl^3\ m^2}.$$

In this equation p and m must be assumed, when either l or N may be assumed and the other found.

If the speed of the engine is to be restricted by piston speed instead of revolutions per minute, let S = maximum piston speed per minute;

$$\text{then } 2p\ \pi\ \frac{m^2\ l^2}{4}\ \frac{S}{60} = Fv\text{, but } \frac{S}{60} = \frac{2ln}{60} \text{ and } \frac{n}{60} = \frac{vN}{2\pi R};$$

$$\therefore\ N = \frac{2FR}{m^2\ l^3 p}.$$ The maximum horse-power will be $HP = \frac{Fv}{550}$.

Constants.—The co-efficient of rolling friction ϕ_1, as determined in the experiments on the Thames River Draw-bridge, was .00257, or in round numbers .003. The co-efficient Φ_2 for collar friction may be given the ordinary values for sliding friction. In the formula for friction of gearing, $\Phi_3 = 0.20$ and $a = \frac{1}{2}$ for cycloidal and three-quarters for involute teeth. In the formula for effect of wind pressure, p may be taken as the pressure due to a wind velocity of 30 miles per hour, which it is believed will be ample.

A good thumb rule for power to operate draw-bridges would be to assume a co-efficient of rolling friction large enough to allow for all resistances. In case of careful construction, a co-efficient of .01 would be ample. For cheap or hasty work .015 should be allowed. To find the horse-power required let W = weight of bridge, v = maximum velocity per second at circumference of rack, then $HP = \frac{.01 \text{ or } .015\ (Wv)}{550}$.

DISCUSSION.

THEODORE COOPER, M. Am. Soc. C. E.—The Second Avenue Draw-bridge in New York City is operated by hydraulic power, operating through wire ropes running over sheaves and attached to the axis of the turn-table wheels. Diameter of track circle, 26 feet. Points of attachment of hauling ropes are on a circle of 27 feet 4 inches diameter. There are four rams, 9 inches diameter of plunger and 6 feet long. Two rams open and two close the draw. The weight upon the track circle is as follows:

	Pounds.
Shop weights of iron and machinery	638 263
Rails, spikes and handrail	36 750
Ties, footwalks, etc	204 987
Total weight	880 000

Soon after the completion of the bridge, experiments were made to determine the frictional resistance. It was found that after the bridge was started from the position of rest, 100 pounds on the rams would keep it in motion, 100 pounds on the two rams = 12.723 pounds. This by means of the sheaves was reduced to 3 344 pounds acting at the circumference of the wheel, or $\frac{3\ 344}{880\ 000} = .0038$, or 3.8 pounds per 1 000, as the force necessary at the circumference of the wheels to overcome all the resistances, including rolling friction, friction of the collars, rams and sheaves. This is almost identical with the amount found by the authors for the Thames River Draw-bridge.

As the estimated force necessary to overcome the inertia for a speed sufficient to open the draw in one minute was 5 400 pounds applied at the center of the track, we needed for the friction and inertia 8 744 pounds, which required 260 pounds on the rams. The rams, accumulators, etc., were designed for a pressure of 400 pounds, and are usually worked to about 350 pounds, in order to have a surplus for wind resistances. The Watertown tests for 1881 give the following results of experiments on the rolling friction of 1-inch steel rollers between cast iron plates under pressures per lineal inch of rollers varying from 1 250 to 3 750 pounds.

Co-efficient of friction—minimum, .00256; maximum, .01212.

Two-inch steel roller, tested in same manner and same pressures.

Co-efficient of friction—minimum, .00125; maximum, .00675.

Four-inch cast roller with pressures from 1 250 to 6 250.

Co-efficient of friction—minimum, .00175; maximum, .0040.

Same under 10 000 pounds pressure per lineal inch.

Co-efficient of friction—minimum, .0045; maximum, .0050.

A French experiment with a 4-inch iron roller loaded with 200 to 370 pounds per lineal inch gave—

Co-efficient of friction—minimum, .003; maximum, .012.

Same sprinkled with fine sand—

Co-efficient of friction—minimum, .026; maximum, .048.

By William H. Burr, M. Am. Soc. C. E.—Although draw-bridge construction on a scale requiring a reasonably accurate determination of the power to be used in operating the turning and locking machinery is happily comparatively infrequent, yet the results of the investigation made by Messrs. Boller and Schumacher are of great practical value. Essentially nothing of the kind has heretofore been done, and while the details of their apparatus are not shown, it may be confidently assumed that nothing was omitted or neglected in connection with it that could conduce to greater accuracy or completeness than that attained. All features of the computations required in the interpretation of the experimental data satisfy the most exacting conditions, and engineers may congratulate themselves on at last possessing something in connection with swing-bridge resistances on which quantitative deductions entitled to a high degree of confidence can be based.

In determining the steam engine capacity for a given draw or swing bridge, it will not be necessary to divide the resistances, except to give a separate expression to that of the wind, nor to compute the moment of inertia of the structure in the detailed manner given in the paper, although the degree of refinement exhibited is strictly appropriate to the purposes of the investigation. By the aid of the total co-efficient of friction, or what may perhaps be more accurately termed the co-efficient of internal resistance of the structure, some very simple expressions for the power required to operate a draw can be easily written. For this purpose it will be assumed that in opening the draw the acceleration of motion (*i. e.*, the velocity gained each second of time) is constant until the highest permissible velocity is reached; after which the maximum velocity remains constant until retardation begins, which latter is taken to be constant until motion ceases; and these assumptions express the actual conditions as nearly as, or nearer than, any other that can be made. A swing span square crossing requires the structure to be turned through 90 degrees in being opened, and the same in being closed. It is not practicable to assign the exact limits to the periods of acceleration, uniform motion and retardation which comprise the total time necessary for the complete 90 degrees of motion, but it is not far wrong to assume that the power of the engine is exerted over one-quarter of the 90 degrees in producing the uniform acceleration which culminates in the maximum velocity, and that retardation takes place over the last fifth of the same 90 degrees, leaving fifty-five hundredths of the path for uniform motion. The wind considerably affects those proportions, but I have frequently observed approximately those just given.

Following the authors of the paper as closely as possible, the notation will be:

W = total weight of the structure in pounds, including turntable, rollers, etc.

R_1 = radius of drum in feet = $\frac{1}{2} D$.

f = acceleration in feet per second of extreme end of the structure.

r = retardation in feet per second of extreme end of structure.

v = maximum velocity in feet per second of extreme end of structure.

l = total length of structure in feet.

w = transverse width of bridge between truss centers.

α = maximum angular velocity $= \frac{2\,v}{l}$.

t = total time in seconds required to open bridge.

g = 32.2 approximate constant for gravity.

Under the conditions assumed the extremity of the bridge would pass over the distance $\frac{\pi\,l}{4}(0.55 + 2 \times 0.25 + 2 \times 0.2) - 0.3625\,\pi\,l$ in the time t, if the velocity were uniform.

$$\therefore v = \frac{0.3625\,\pi\,l}{t}; \text{ or, } t = \frac{0.3625\,\pi\,l}{v} \quad \ldots\ldots\ldots\ldots (1)$$

$$\text{or, } \alpha = \frac{0.725\,\pi}{t}; \text{ and } t = \frac{0.725\,\pi}{\alpha} \quad \ldots\ldots\ldots\ldots (2)$$

According to what has just preceded, the acceleration f will exist during the time—

$$\frac{2 \times 0.25}{0.55 + 2 \times 0.25 + 2 \times 0.2}\,t = \frac{50}{145}\,t = 0.345\,t.$$

$$\text{Hence: } f = \frac{v}{0.345\,t} = \frac{1.05\,\pi\,l}{t^2} \quad \ldots\ldots\ldots\ldots (3)$$

$$\text{And the angular acceleration } \alpha = \frac{2.1\,\pi}{t^2} \quad \ldots\ldots\ldots (3a)$$

In the same manner the retardation will be—

$$r = v \div \frac{40}{145}\,t = \frac{v}{0.276\,t} \quad \ldots\ldots\ldots\ldots (4)$$

In determining the moment of inertia of the structure about the vertical axis of revolution, it will be sufficient to consider the trusses, lateral bracing, floor system and track as a homogeneous prism with length l and width w. This portion of the weight is, for all practical purposes, $\frac{5}{6} W$. Its moment of inertia will then be—

$$I^1 = \frac{\frac{5}{6} W\,(w^2 + l^2)}{12\,g} = \frac{5\,W\,(w^2 + l^2)}{72\,g} \quad \ldots\ldots\ldots (4)$$

The moment of inertia of the drum, rollers, etc., will be determined by considering the weights of those parts concentrated at the distance R_1 from the axis. Hence that moment will be—

$$I' = \frac{1}{6} \frac{W R_1^2}{g} = \frac{W R_1^2}{6 g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Hence the total moment of inertia is—

$$I = I + I' = \frac{W}{72 g} (5 w^2 + 5 l^2 + 12 R_1^2) \quad \ldots\ldots\ldots\ldots\ldots (6)$$

The velocity of the extremity of the draw at the beginning of the last unit of time (*i. e.*, the last second) of the period of acceleration is—

$$v - f = v \left(1 - \frac{1}{0.345\, t}\right).$$

Hence the path passed over by that point during that unit of time is—

$$v - f + \tfrac{1}{2} f = v \left(1 - \frac{1}{0.69\, t}\right).$$

The corresponding path under the rollers is—

$$\frac{2 R_1 v}{l} \left(1 - \frac{1}{0.69\, t}\right) = \frac{0.725\, \pi R_1}{t} \left(1 - \frac{1}{0.69\, t}\right).$$

Hence the work performed during the same unit of time in overcoming friction is—

$$F = \frac{0.725\, \phi\ W \pi R_1}{t} \left(1 - \frac{1}{0.69\, t}\right) \quad \ldots\ldots\ldots\ldots (7)$$

If the unbalanced wind pressure of p pounds per lineal foot is taken to act on the entire length of one arm and constantly normal to the longitudinal axis of the structure, the center of that total pressure will be at the middle point of the arm, and the work performed in overcoming it during the same unit of time to which equation (7) applies, will be—

$$A = \frac{p\, l}{2} \times \frac{v}{2} \left(1 - \frac{1}{0.69\, t}\right) = \frac{0.0906\, \pi\, p\, l^2}{t} \left(1 - \frac{1}{0.69\, t}\right) \quad \ldots\ldots\ldots (8)$$

Finally the work expended in accelerating the motion of the structure is, by equations (3*a*) and (6)—

$$E = \frac{I \alpha^2}{2} = \frac{0.0306\, \pi^2\ W}{g\, t\, 4} (5 w^2 + 5 l^2 + 12 R_1^2) \quad \ldots\ldots\ldots\ldots (9)$$

The maximum amount of work, therefore, to be performed in a unit of time (*i. e.*, one second) is—

$$F + A + E = \frac{\pi}{t} \left(1 - \frac{1}{0.69\, t}\right) (0.725\, \phi\ W R_1 + 0.0906\, p\, l^2) + \frac{0.0306\, \pi^2\ W}{g\, t\, 4} (5 w^2 + 5 l^2 + 12 R_1^2) \quad \ldots\ldots\ldots\ldots (10)$$

The horse-power required by the engine will then be—

$$H P = \frac{F + A + E}{550} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

After the greatest velocity v is acquired, this work of the engine will immediately drop to that required to overcome the friction and wind resistance. The brake power required in retardation over and above that part of the latter caused by the friction, can readily be expressed by the same general method as the preceding, but it has no special practical value.

The investigations of Messrs. Boller and Schumacher show that for the Thames River bridge, the total co-efficient of friction was 0.004, very closely. Hence other data from the same structure are—

$l = 500$ feet $\quad w = 28\frac{1}{3}$ feet $\quad W = 2\,400\,400$ pounds.

$t = 180$ seconds $\quad R_1 = 16$ feet $\quad p = 100$ pounds.

By equation (6) the moment of inertia becomes—

$$I = 1\,301\,496\,880 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

By inserting the remaining data, as just given, in equation (10)—

$$F + A + E = 1\,928 + 39\,207 + 27 = 41\,162 \ldots\ldots\ldots\ldots (13)$$

$$\text{Hence, } H.\ P. = \frac{41\,162}{550} = 75 \text{ horse-power} \ldots\ldots\ldots\ldots (14)$$

It is interesting and important to observe that only about 5 per cent. of the total power is required to develop the acceleration and overcome the internal resistance; the remaining 71 horse-power is entirely applied to overcome the wind pressure. This latter has been taken at 10 pounds per square foot on an assumed area of 10 square feet per lineal foot of structure. These computations demonstrate the importance of the part played by wind pressure in the operation of swing bridges. The co-efficient of friction found by Messrs. Boller and Schumacher undoubtedly applies only to the unusually excellent grade of workmanship which characterized the Thames River construction. For all ordinary cases it will probably be best to make $\phi = 0.01$, for rim bearing tables. But even with this increased co-efficient the horse-power required for friction and acceleration in the present case would be but $\frac{4\,820 + 27}{550} = 9$.

A comparatively small amount of power is, therefore, required to operate a swing bridge if there is no wind or if the wind pressures on the two arms are balanced.

If P is the total steam pressure on the piston of the engine, s the stroke and n the number of revolutions for the same unit of time as that to which α applies, and if steam follows the piston the full stroke, as the greatest wind pressure may require, the total work of the engine per unit of time will be $2\,n\,s\,P$. The work of the wind pressure and friction during the same time will be $\left(\frac{p\,l}{2} + \frac{l}{2\,R_1} + \phi\,W\right)\alpha\,R_1$. Hence motion will begin to decrease and the engine will eventually be stalled if $P < \left(\frac{p\,l^2}{4\,R_1} + \phi\,W\right)\frac{\alpha\,R_1}{2\,n\,s}$.

Precisely the same general method as the preceding may be applied for the determination of power required for swing spans under any division of acceleration, uniform motion and retardation periods.

Leffert L. Buck, M. Am. Soc. C. E.—Although this paper is in itself a valuable contribution, its value will be greatly increased should its discussion bring out the results which others have obtained from ex-

periments in a like direction, as it should do. The literature of the draw-bridge would then be increased in a direction in which it is at present greatly lacking. It might be well to request others to send in to the Society such data upon this subject as they may possess. It appears to me that the greatest value of the present paper lies in the fact that it gives the total resistance to motion of the draw, with the exception of that part due to inertia, which, in this case, was the force used in measuring the remaining resistances. The aggregate of the resistances is what we want most.

The authors of the paper are undoubtedly correct in saying that it is difficult to estimate the resistance of friction of the rollers against the live ring. But this is not necessary, unless for the purpose of trying to devise some substitute for the live ring. It is doubtful if the weight of the draw is directly an element of the pressure against the ring. The angle of the curve is so far within the limiting angle of friction that no weight could press the wheels out. It is imperfect workmanship that causes them to press out against the ring. If the workmanship was theoretically perfect and the wheels were placed either outside or within their true position, their tendency would be to roll into it. The coefficient derived from the experiment of attaching a rope to the end of one arm of the draw, passing the other end over a sheave and suspending a weight to it, even though the apparatus was crude, probably does not exceed that arrived at by the other experiment by more than the resistances of the sheave and of inertia, which last was in this case a resistance, especially if we consider that the starting friction is greater than the moving friction.

F. W. Skinner, M. Am. Soc. C. E.—I have enjoyed and appreciated the paper of Messrs. Boller and Schumacher, which is valuable for the careful and laborious analysis and computations in a subject very little, if any, discussed or considered in a manner available to engineers. The theoretical character of the paper and its exclusively mathematical value make it difficult to discuss off-hand. Perhaps, too fine drawn for the practical working conditions of such cases as the one considered, but may be none the less valuable for the analysis of the problem and to indicate methods and steps of its general solution.

I agree with the other speakers that it needs to be reinforced with parallel investigations, and trust that some of our practical bridge engineers will present further data of similar character. To pass from the grave to the ridiculous, without comparison with the important subject of the paper, I may mention a humorous example of resistances in swing bridges that is suggested by recollections of patent highway swing bridges that I once knew to be, and probably still are, somewhat numerously manufactured by an Ohio bridge company. They were light country bridges, of perhaps 35 to 45 feet span, over canals. The swing was center bearing, and had a tall pivot with double cylin-

drical or conical walls, between which double spiral springs (of, perhaps, 1-inch square steel) were coiled in opposite directions, so as to oppose the swinging of the bridge in either direction from its axis. On each side of the bridge, along its lower chord, a longitudinal wooden buffer timber was placed, against which passing canal boats struck violently, forcing the bridge open, and scraping their way through, after which the spring vibrated the bridge until it finally stopped in a closed position.

W. H. Breithaupt, M. Am. Soc. C. E.—This paper is a valuable contribution to draw-bridge literature in that it gives concise data on something that has mostly been a matter of rough approximation only, and in this goes into an almost wholly new field. I cannot add much directly in the line of the subject of the paper, but might give an illustration of the obstruction by varying wind pressure on opposite ends to the turning of a long draw span, as spoken of in the paper. In the Fort Madison bridge, over the Mississippi River, C. S. F. & C. Ry., there is a 400-foot draw. This bridge carries a railway and a wagon road on either side on outer brackets. As it was desirable to cut off the view of trains from the roadways, 6 feet 6 inches of open slatwork screens were interposed between the roadways and the railway track. In figuring on the turning machinery, the weight of the draw span, which is a very heavy one, and regular wind surfaces, were allowed for. It was found, however, that the screens referred to, adding 3 feet or more per running foot to the effective wind surface as they do, made it extremely difficult to operate the draw during a wind, so that the turning machinery had to be reinforced.

In heavy draw-bridges it is of great importance to have the distribution of dead load over the turn-table uniform; to have the drum and the cross girders which transmit the weight to it, or to it and the center, stiff and rigid ; to have the wheel track perfectly level and on an unyielding support, and to have all machine work accurately done. I see no good reason why the turn-table should always be preferably rim bearing only. When a large part of the weight can feasibly be transferred to the center it is well to do so, as it can from there more easily be distributed with uniformity to the masonry, and there is the further advantage of shorter traverse of moving parts there, and relatively greater accuracy obtainable in the working surfaces, resulting in greater ease of turning.

Concluding Remarks by the Authors.

The reason the total resistance to motion of the draw was not given, as suggested by Mr. Buck, was because this resistance is made up of the constant resistance friction and the variable resistance necessary to overcome the inertia. The determination of the former was the main object of the paper ; the latter, varying with each velocity, may be found readily for any given velocity.

The method employed by Mr. Cooper upon the Second avenue bridge is practically the same as the last method used by us as a check upon the first method. It is gratifying to see the co-efficient determined by Mr. Cooper agrees so very nearly with that determined for the Thames River bridge.

In figuring the moment of inertia Mr. Burr assumes $\frac{5}{6}$ W in the trusses, flooring, etc., and $\frac{1}{6}$ W in drum. In the Thames River bridge the weights are nearly—

Trusses, flooring, etc.	2 185 000
Drum, etc.	215 000
	2 400 000

Which is nearer $\frac{1}{11}$ W and $\frac{1}{12}$ W.

Substituting these values then—

$$I = \tfrac{1}{3}\ 2\ 185\ 000\ (250^2 + 14^2) + 215\ 000 \times 16^2$$

$$I = 45\ 717\ 170\ 000,$$

which is nearly (within 5 per cent. of) the figure previously found by the longer and more accurate method, *i. e.*:

$$I = 43\ 011\ 086\ 348.$$

The computation for wind pressure as assumed by Mr. Burr is probably excessive. By Smeaton's formula, given by Trautwine, 10 pounds per square foot corresponds to a wind of nearly 50 miles per hour. This wind is considered to blow entirely upon one end of the draw, which would require a severe hurricane blowing, and in that case the draw would not be required to be opened ; 30 miles an hour would be ample to allow, which would correspond to 4.5 (say 5) pounds per square foot. The area of 10 square feet to 1 foot of draw is a large estimate for the Thames River bridge. But allowing this, then

$$F = 1\ 920 \quad \text{H. P.} = \frac{22\ 470}{550} = 40$$

$$A = 19\ 609$$

$$E = \frac{941}{22\ 470} \qquad \frac{19\ 609}{22\ 470} = 87 \text{ per cent.}$$

Forty horse-power is somewhat nearer the capacity of the engines than 75, as determined by Mr. Burr. There are two oscillating engines, 10-inch diameter, 7-inch stroke, steam full stroke, say at 40 pounds pressure, and at 150 turns per minute.

$$\text{The horse-power} = 4\,\frac{40 \times 7 \times 78 \times 150}{12 \times 33\ 000} = 33.$$

The analysis of Mr. Burr, it is very clear, shows from how many different points this question may be viewed. It would be well if the profession could have more discussions like this one. We desire to express our thanks for the kind manner in which this paper has been received and discussed, and hope that now the field has been opened additional data upon this subject may be developed.

Our object in dividing the resistances in Part I of the paper was to arrive at the rolling friction. In Part II we considered division of the resistances necessary, because the frictional resistances are constants, determined by experiment; wind is practically arbitrary, and inertia varies with the time required to operate the draw. We have given a very simple formula at the end of Part II, which is:

$$\text{Maximum horse-power} = \frac{.01 \text{ or } .015\,(W\,v)}{550}.$$

The constants are more than sufficiently large to swing the bridge, but in case of a necessity for an allowance for wind and rapid operation of draw they should be doubled or trebled. This is made evident by the fact that in swinging the Thames River draw the steam pressure under ordinary circumstances was about 40 pounds, which, with the dimensions and revolutions of the engines, gave about 30 horse-power; and substitution in the above formula, considering maximum velocity at end of draw 3 feet per second, gave about 10 horse-power.

The information as to the usual periods of acceleration, uniform velocity and retardation in practice are very valuable, and will give practical value to the general formula under the head "Acceleration" in Part II. Mr. Burr's analysis differs from our own chiefly in the fact that he uses angular acceleration where we use linear, and takes as his unit of moment of momentum the unit of mass where we use the unit of weight. The suggestion to consider the span as a homogeneous prism was anticipated in Part II, under the heading "Inertia." The value $\phi = .01$, suggested by Mr. Burr, was recommended by us at the end of Part II.

The necessity for considering the friction of the rolls against the live ring is evinced by the fact that the spider rods which bear the collars against which the rolls press, and which support the live ring, occasionally break; and it is impossible, with the most accurate machinery with which the rolls and track can be made, to secure theoretical accuracy, under which condition only there will be no lateral tendency of the rolls. Even if machinery could be devised to procure theoretical accuracy at first, unequal wear and settlement of pier and track would soon destroy the theoretical accuracy.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—December, 1891.)

RED ROCK CANTILEVER BRIDGE.

FOUNDATIONS.

By S. M. ROWE, M. Am. Soc. C. E.

WITH DISCUSSION.

As a formal report has already been made to the Atlantic and Pacific Railroad Company, through its General Manager, D. B. Robinson, the purpose of this paper will be to outline the history of the construction of the bridge; the causes that led to its construction, and to give all facts that may be of interest to the civil engineer and the scientist. The original line of the Atlantic and Pacific Railroad, when constructed in 1883, was laid in such a way as to skirt and traverse the valley of the Colorado River for a distance of nearly 8 miles, finally crossing that river about 2½ miles south of what is now Needles Station, on the California side. This point of crossing is 18 miles south of Fort Mohave, near which is fixed the southernmost point of the State of Nevada. Ten miles south of the original bridge, the river valley narrows into what is known as Mohave or Needles Cañon; so named from a small but rugged

range of volcanic mountains on the Arizona side. At the head of this cañon is the site of the Red Rock Bridge.

The Colorado River rises in southern Wyoming, draining western Colorado, eastern Utah, a small portion of southern Nevada and eastern California, and about three-quarters of the Territory of Arizona. The drainage area is about 230 000 square miles, most of which is mountainous, consequently its water is always turbid, and the quantity of silt held in suspension and carried forward at the same velocity as the water, is very considerable, amounting, as determined by precipitation of numerous samples, to 1.56 grains per cubic inch of water. The silt when thoroughly dry is found to weigh 59.95 pounds to the cubic foot.

While most of the distance through which the river flows is across a desert, almost rainless (except on rare occasions in which storms of great violence occur), its tributaries rise mainly on the west slope of the Rocky Mountains, where heavy rains are frequent and where the snowfall is sometimes quite heavy. Large accumulations of snow suddenly set free by a sudden thaw or by heavy rain, owing to the great declivity of the land, will cause violent floods and correspondingly severe attrition both on banks and river bed, by which earth, sand, gravel and boulders will be carried forward.

To form some idea of the amount of silt carried forward, take the fine silt above mentioned, much of which is so fine that it requires hours or even days to precipitate it. In the high water of 1884, taking the record as given by Chief Engineer W. A. Drake, who estimated the maximum velocity at 8 miles per hour (which seems from more recent observation to be within bounds), and an approximate area of river at 48 500 square feet; then assuming 5 miles per hour as the mean velocity, giving 384 000 cubic feet of water per second, the 1.56 grains per cubic inch would give 7 900 000 cubic yards of earth, or 6 400 000 tons at 1 620 pounds per cubic yard for each twenty-four hours of that flood. Then taking 50 000 cubic feet per second (about the mean for the year) as the mean amount of water passing, we would have about 160 000 000 cubic yards per day. Add to this the movement of the heavier sands and gravel, which is constantly going forward (though at less velocity), both by the movement of sand bars in the river bed and from cutting away of the banks, and it will be seen that this river is a powerful excavator as well as disintegrator. The Grand Cañon stands as a notable monument of this. In the case in hand, the line, as built, was subject to the latter mode of attack, and from the

great depth to which the cutting extended (20 to 40 feet), it was found impracticable to protect the road-bed.

The valley of the Colorado River, for a long distance above the Needles Cañon, is not more than 12 to 20 feet above the ordinary water level, and has been traversed to its bounds by the river channel, and no doubt will continue to be so. Indeed, quite radical changes are taking place yearly. The greatest flood occurs in June, and is usually due to melting snow in the Rocky Mountains; but rises of 3 to 5 feet may occur at almost any time in the season, and are liable to come with little or no warning, the source being so far distant that they do not correspond to the local weather indications in the least. The declivity of the river from Fort Mohave to the Needles Cañon, and for a mile or two into the cañon, is about $1\frac{4}{10}$ feet per mile, though probably greater below that point, as the river is quite narrow where it cuts through the Needles Range.

The volume of the river varies from a minimum of 5 000 cubic feet per second to probably 500 000 in extreme high water (elevation 469.6 to 500.5). While the danger from this was not unforeseen by the locating engineer, Mr. Lewis Kingman, its gravity was not fully appreciated. The line finally adopted was surveyed by him in 1880, and the cost was estimated at $666 220, while the line then built was estimated at $586 554, making a difference of $79 666 in favor of the latter. The line *via* Red Rock, then designated as the lower crossing "C," was identical with the point of crossing selected by the Kansas Pacific engineers in 1867, and also by the Atlantic and Pacific engineers in 1871.

The experience of the six years since the completion of the original line was such as to again force the attention of the management to the necessity of some effective remedy, and a joint letter from William B. Strong, President of the Atchison, Topeka and Santa Fé Railroad Company, and E. F. Winslow, President of the St. Louis and San Francisco Railroad Company, was issued January 30th, 1888, directing A. A. Robinson and James Dun, Chief Engineers of the respective companies, to examine with regard to what improvement was necessary to put the line in proper condition.

Leaving out many minor matters relative to the improvement of bridges, buildings, track, equipment, etc., we quote from the report made by these two chief engineers representing the joint ownership of the road, so much as relates to the question in hand.

Colorado River Crossing.

"One of the most serious questions which we have to report upon is the crossing of the Colorado River. We find upon examination, that the present line is unsafe; that a portion of it is below recent high water mark, and all of it is subject to the encroachment of the Colorado River, so that it will be impossible upon the present line to give you any reliable figures as to the present or future cost of maintaining the present line and bridge, except to say that we know it will be very great.

"The estimate for the change of line and the maintenance of the present crossing, as shown on Exhibit 'C,' is based on the highest water known, but it is possible that the data are insufficient to establish the true high water mark; in which case there would be a constant element of danger from this source, which is entirely eliminated by adopting the Red Rock Crossing line. During the last six months the river has encroached upon the line about midway between Powell and the present bridge, threatening the line for more than a mile. This action of the river is undoubtedly only the beginning of the working over of the bottom into a new channel, as is customary with rivers of this class.

"While the expenditure of forty or fifty thousand dollars at this time might protect the road during the year 1888, it is more than likely that the same expense would have to be incurred during 1889, and indefinitely thereafter, with the more than probable danger of having our line cut in two entirely, and traffic suspended several weeks or months. Under this condition of affairs we see only one remedy, that is, to place the crossing of the Colorado River upon a safe and permanent basis.

"Two plans suggest themselves; one of which is to build the line directly across the valley, instead of following up the middle of the valley as by the present line, and building your track upon this bank from the present bridge to the bluff on the east side of the valley; thence along the foot of the bluffs and connecting with the present line near Powell. This plan will enable us to use the present bridge for some time to come, by lengthening it 500 or 600 feet. The road could be built upon this new location for about $160 000. This, however, would leave us with the river to fight, to maintain the channel under the present bridge. This expense of maintaining the channel from year to year will continue, and would undoubtedly amount to several thousand dollars each year. In addition to this, the character of the Colorado River is such that it is almost impossible to maintain a channel at any fixed point. The river is a navigable one, and the bridge is built upon a low grade line. The draw span is not serviceble, because the main channel of the river passes under the bridge several hundred feet from the draw. As a result of this you are paying a steamboat company now

plying the river, $500 per month, because of the obstruction of the river by our bridge, and are unable to let boats pass. This is a dangerous situation, and leaves the gate open for any enterprising individual to buy an old rotten tub, and dispose of it to advantage by blackmailing the railroad company. The amount which we are now paying this steamboat company, namely, $6 000 a year, represents the interest on $100 000 at 6 per cent. The chances are, as we have intimated, the day is not far distant when more will be demanded.

"These considerations have led us to the conclusion that we can only say that the best course to pursue is to reconstruct the line for about 13 miles, crossing the river on a high grade line at the Red Rock crossing, about 9 miles below the Needles Station. This would put the line forever upon a safe basis and stop the further expenditures for protecting your road-bed and bridges against the river and for damages to navigation, and, in our judgment, is the only safe and wise course to pursue.

"We could not, of course, construct this bridge in time for the freshet of 1888; consequently, the expense of maintaining your present line during this season would have to be incurred. What this will be no person can tell until the freshet comes. The work which has already been done, seems to us to have been judiciously done. The money has been well expended, and is all that can be done until the high water season develops what further expenditures are necessary.

"Exhibit 'C' herewith gives you an estimate of the cost of reconstructing this 13 miles of road and the permanent bridge, and also the cost of rebuilding and putting in a permanent structure on the present bridge site and changing the line from Powell as above indicated.

"We would recommend that in doing this work, the grading be contracted for completion by the 1st of August; that the contract for the bridge be made for delivery on the 1st day of November, and that the track be laid to the bridge by the middle of September; that the work upon the masonry for receiving the superstructure be completed on November 1st, and that the bridge be erected and ready for traffic by January 1st, 1889. It is possible that the expenditure might be delayed, and the interest on the investment saved for three months more, but considering the difficulties liable to be encountered, the distance of the undertaking from market, and the extreme heat* which prevails in that climate, we think the plan should be laid as above indicated, to make sure of having the structure completed in ample time for the freshets of 1889. Thirteen miles of new steel will be required on this work.

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*Heat; maximum atmospheric 124 degrees Fahr.; of sand, 135 to 145 degrees, and metal 165 degrees; minimum atmospheric, + 26 degrees Fahr.—*Rowe.*

NOTE.—During the summer of 1888 the river encroached both above and below the bridge, and with 3 feet more water, the line would have been lost and the river channel would have crossed the line about 3 miles east of the bridge. Luckily the summer flood was less than usual; notwithstanding this, the track was saved only by an expense of over $500 per day for over a month.—*Rowe.*

"We are sorry, indeed, to have to recommend to you the expenditure of so large a sum of money. We are certain, however, that a great mistake has been made in locating the line upon its present road-bed, and so we believe that the only course now open for this company is to rectify this mistake; that to continue to maintain the road-bed in the present position will only result in loss and disaster, which will be many times greater than the work of placing the line upon the proper location. There is no question whatever in our minds, but that this is the proper course to pursue."

The river bed had been examined systematically by Louis Trainor, Engineer of the Southern Pacific Railroad Company, in 1881, at which time that company was engaged in building eastward from Mojave. The maximum depth at midstream to reach rock was made about 30 feet, and it was assumed that by using a 400-foot span at midstream, the depth of foundation would not exceed 24 feet.

Commencement of Survey.—On July 2d, 1888, the writer received orders to make a survey and estimate of the proposed line crossing at Red Rock, and on the 4th walked over a portion of the line, making 15 miles, and on return to Needles Station at 6 P.M., found the thermometer standing at 122 degrees Fahr. This is simply mentioned so that the conditions may be to some degree appreciated by those unacquainted with the climate of that locality. The surveys were prosecuted vigorously, and only the river soundings remained to be done; these only awaiting arrival of appliances ordered and the subsidence of the summer flood, now about passed. On September 14th, in response to urgent request from the management, the following report was transmitted through the General Superintendent, A. A. Gaddis:

Preliminary Report of Survey of Red Rock Line.

"Albuquerque, N. M., September 13th, 1866.

A. A. Gaddis,

General Superintendent.

I submit, in accordance with your directions, contained in your letter of July 2d, a report of survey, estimate and map (Plate CXI) of the proposed change of line, and of crossing of the Colorado River at Red Rock, as recommended by Messrs. Robinson and Bond, in their report submitted to Presidents William B. Strong and Edward F. Winslow, in February of the present year.

The Line.—Commencing at Station 14 571 + 41, a point 3 050 feet west of mile-post 562, or mile 562.6 of the main line of the Atlantic and Pacific Railroad; we run immediately across the Sacramento wash, and supporting on the side hill we reach the river immediately opposite

to what is known as Red Rock. Crossing the river, we are obliged to curve at the earliest practicable point, to avoid precipitous hills, using 9 degrees curvature, being the lightest practicable. Thence we run up the edge of the river valley, keeping at all points above extreme high water marks, and avoiding possible attack from the river, and, at the same time, keeping the grade line sufficiently above the beds of the washes from the hills west to insure the passage of water under the track. We connect again with the present main track just west of the present bridge, at Station 312 + 84.5, at a distance by present line from the point of starting of 54 156.9 feet, having run by the new line 70 184.6 feet, making an increase in distance of 3.036 miles. A preliminary profile is herewith submitted, showing curvature and grades, hastily laid, from which to deduce approximate quantities.

Before proceeding with work the line should be slightly shifted in several places, and the grades revised. The approximate quantities given I feel quite sure will be somewhat reduced in construction, but will serve as a basis of estimate. The desire to save any unnecessary expense and not to delay the transmission of this report impels us to submit the matter in a less perfect state than is desirable. Most, if not all the items are intended to be above, rather than below what the actual cost should be.

Grading.—In estimating the cost of grading, I assume that 20 cents per cubic yard for waste and borrow; 26 cents for excavation hauled; 64 cents for loose, and $1 for solid rock excavation, and 1½ cents per cubic yard hauled 100 feet, will be a fair estimate for this work. The quantity of solid and loose rock will be small.* In view of the approach of the favorable season, these figures can probably be shaded somewhat. Water is convenient and supplies in easy reach. A detailed statement of quantities is appended.

Pile and Trestle Bridges.—Owing to the numerous washes making into the river, particularly on the west side of the river, the amount of pile bridging will be large. I think there is no place (case) where piles cannot be driven. It will require about eighty pile bridges, varying from 30 to 345 feet in length, with a mean height of 10 feet in the clear. These I would estimate at $6 per linear foot.

Iron Bridge at Red Rock.—It was intended to make full and careful soundings of the river bed at the line of crossing, but as it required at the present stage of water, expensive equipment and considerable time, it was deemed best to rely for preliminary estimate on the soundings made by Louis Trainor in 1881. These soundings were made under very favorable circumstances, the river being, at that time, nearly 6 feet lower than at present.

As the rock in question, on which it is proposed to build, is a volcanic "breccia," forming a dyke across the river at this particular place, the

* Most of the material to be moved is sand and loamy clay, and in no case marshy.

depth is liable to great variation, even in the length of the pier (about 60 feet); and I would advise careful examination by sounding before the line is fully established across the river. A slight shifting may be very advantageous and save much expense. The river is gradually falling, and where a great volume of rapid flowing water was passing two months ago, to the depth of 20 feet, is now a mass of sand and mud with not over 8 feet of water at the deepest point.

Plan of Bridge.—I submit plans for piers for the channel span, which may prove too light. I will examine them later with reference to this. They will be exposed to the force of water and drift only, as there is no ice in the Colorado River; but at extreme high water this will be considerable, as the water has at some time—years ago—attained a depth of from 50 to 60 feet, and a very high velocity.

Stone for Masonry.—I think the Chino stone should be used and the masonry should be first-class, laid with the best cement. The English Portland can be bought, I understand, at San Francisco, for $3.50 per barrel; adding freight, 50 cents, would deliver it at the Needles for $4, being about as cheap as domestic cement could be laid down at that point, and it will, I think, be most economical and give the best results for the cost.

After October 1st, on and until the May following, the river will be low, and I think there will be no serious obstacles to prevent the sinking of cribs for the two channel piers and the erection of the falsework for raising the bridge. The work cannot be delayed after the 1st of October or possibly a month later. From May 1st until about the present time it is impracticable to undertake such a work.

Relating to Erection of the Bridge.—I would advise the building of the line from the Needles south to the bridge site, as material can be delivered on west side of the river more conveniently.

In relation to the 396-foot channel span there might be a saving by using a shorter span, but the manner in which the channel shifts would seem to indicate it as being safer than a shorter span. From my observation of the river during high water, I think the span, as located, will always cover the main channel.

Superstructure.—By courtesy of the Keystone Bridge Company I am able to give their maximum figures for the erection of the superstructure of the bridge and the iron trestle approaches (Plate CXII). The quantities given for the masonry are the contents of the piers as shown by plan. The price assumed at $12 per cubic yard seems high, but I doubt whether Chino stone can be put in any cheaper, as it is as hard and nearly as heavy as granite.

A sketch for a caisson is submitted (Plate CXIII), showing a section across a pier and manner of weighting it down (Plate CXIV). My estimate for 886 cubic yards of beton is for this purpose and to fill the inner space when the pier is completed. As you will perceive, this will increase the

base of the pier at least 3 feet on all sides, and obviate largely the necessity of riprap. I think the weighting in this manner is the best, as it strengthens, and at the same time renders the wall of the caisson watertight.

GRADING 13 $\frac{3}{10}$ MILES:			
Embankment	261 346.8 cubic yards, at 20 cts.,	$52 269 36	
Excavation hauled	72 968.6 " " " 26 "	18 971 84	
Loose rock excavation	2 332.7 " " " 64 "	1 492 93	
Solid rock excavation	6 529.4 " " " $1	6 529 40	
Excavation hauled 100 feet	534 778.9 " " " 1½ "	8 021 69	
Total			$87 285 22
Pile bridges (78) aggregating 7 185 linear feet, at $6 per linear foot			43 110 00
MASONRY:			
Bridge masonry	4 212.73 cubic yards, at $12,	$50 552 76	
Pedestal masonry	115.22 " " " 15,	1 728 30	
Cement, E. P.	2 165. barrels, " 4,	8 660 00	
Total			60 941 06
FOUNDATIONS:			
Cost of two caissons		$5 826 00	
Labor and machinery sinking foundation		10 500 00	
Labor on other piers and abutments		3 000 00	
False work for bridge		9 500 00	
Total			28 826 00
Beton for filling cribs, 886 cubic yards, at $8			7 088 00
SUPERSTRUCTURE:			
One span, 396 feet, at $160 per linear foot		$63 360 00	
Two spans, 195 feet 8½ inches, at $89 per linear foot		34 818 58	
281 linear feet iron trestle, at $33.60		9 441 60	
Total			107 620 18
Engineering and exigencies			15 000 00
Freight on iron, Albuquerque to Needles, 975 tons at $5.76			5 616 00
Freight on Chino stone, 9 100 tons at $1.42			12 922 00
Then add as per report of Messrs. Robinson and Bond—			
3 miles new steel rails, 60 pounds		$16 060 00	
9 500 new ties, at 72c		6 840 00	
Laying and surfacing 13 miles track		10 000 00	
Total			32 900 00
Grand total			$401 308 46

(Signed) SAML. M. ROWE, Resident Engineer.

Following the transmission of the foregoing report, the location of the line was completed except in the immediate vicinity of the crossing of the river, and requisitions were made for machinery, boat and cable necessary to do the sounding; but in view of the considerable cost of the appliances and in the absence of any knowledge whether the work was likely to proceed, the local management hesitated to incur the expense,

hence this part of the work was delayed until orders were received to let the work. The only thing then practicable was to presume on the correctness of the Trainor soundings, and so let the work, providing for such changes in the bridge structure as circumstances would make necessary. Facing the necessity of prompt action, to be able to finish the bridge before next season's high water (it now being the 20th of November), it was decided to employ Waddell and Jenkins, professional men at that kind of work, and possessing the necessary appliances, to do the sounding. Mr. Jenkins of this firm was promptly on the ground and proceeded to vigorously prosecute the boring. At the first test boring at station 193 + 75, where it had been planned to locate the west channel pier, it was found that instead of 22 feet to bed rock, the drill passed with but little interruption, to a depth of 64 feet. Meantime, the grading was let and well under way and the superstructure and substructure, both let provisionally; the former to the Phœnix Bridge Company, the latter to Sooysmith & Co. On making this discovery, all proceedings under these contracts were stopped and a thorough examination of the river bed was arranged for with the contractors for the soundings. Orders were issued to make a sounding every 50 feet so as to develop a complete section of the river on the line fixed upon. These soundings were completed January 1st, 1889.

When it became apparent that the Trainor soundings were entirely unreliable, and while the new conditions were being developed by the progress of the soundings, various combinations, using various lengths and numbers of spans, were carefully compared and the cost estimated approximately. While the plan seemingly best adapted to all the conditions of the case—that of two through spans of 405 feet each—did not largely exceed the cost of the various combinations of shorter spans, yet two considerations following were regarded as formidable in the opinion of the writer hereof, who at that time filled a subordinate position with relation to this work. In the first place, spans of such unusual length would be heavy and require a much larger amount of metal, and necessitate the maximum amount of masonry, as all the piers would have to be built to the grade line nearly 80 feet above low water, and to proportionately increased dimensions. This for the reason that the depth from grade to high water was insufficient to allow either span to be used as a deck span. In the second place, taking the known history of the river and the depths to bed rock at the point where the pier at midstream

would be, it would be subjected to unusual stress from the great depth of water flowing at the excessive speed of 12 to 14 miles per hour and to a depth of nearly 100 feet. There is good reason to believe that the scour at that point has reached almost to the surface of the rock and that this probably occurred in the nowise extreme flood of 1884. The records of the Trainor sounding of 1881 certainly indicated that a stratum of boulders similar to that found in sinking the caisson at Station 189 + 25, did at that time extend entirely across the whole bed of the stream. By reference to Plate CXVI, it will be seen that the Trainor soundings and those of Jenkins conform very nearly, from the east bank to a point nearly one-third of the distance across the river (Station 190 + 50). The Jenkins soundings, however, found nothing of the kind beyond that point, giving plausibility to the conjecture that the flood of 1884 had removed all beyond the point mentioned. There is nothing above the line of the Jenkins soundings, except sand and gravel and a very few small boulders. Hence, while these objections were not deemed insuperable, yet it was considered best to survey the whole field before deciding. At the time that the chief engineers of the two companies visited the bridge site in February, 1888, the writer being present, suggested that, in view of the formidable character of the stream and the uncertainty as to foundation for piers in the river, the cantilever might be adopted. It was answered, however, that the cantilever was an expensive type of bridge, and if a foundation could be found at moderate depth, the truss would be cheaper.

When the Jenkins soundings had progressed so as to not only demonstrate the utter unreliability of those formerly made, but also to indicate that the foundation for at least one pier toward the east shore could be had at a very moderate depth, the question of the cantilever immediately recurred, and singularly enough, Mr. Waddell, agent of the Phœnix Bridge Company, came in response to a telegram, with a hasty but quite well digested plan for such a bridge (Plate CXV). The span lengths were fixed and orders telegraphed to Mr. Jenkins to thoroughly sound a pier site at Station 189 + 25. While this was being done, the plan of the superstructure was made and checked over simultaneously at Albuquerque and at Phœnixville, by aid of the telegraph, and on December 29th, 1889, a proposition was received from the Phœnix Bridge Company of a pound price as had been provided in the provisional contract, just four days after the matter was taken up.

Numerous soundings were made at and about the proposed site, and by the order of the writer, one center boring was pushed down so as to determine, if possible, the character of the rock. This was done and the depth of the boring into the rock below the surface reported at 5 feet. The rock increased in hardness to such an extent as to prevent further progress with the drill. The depth below low water did not exceed 14 feet in the borings on the outskirt of the site, or a mean depth of 12 feet for the foundation.

The plans for the substructure were prepared immediately and work was at once commenced by Sooysmith & Co., under an amended contract. The specifications were prepared by Consulting Engineer Prof. S. W. Robinson, of the Ohio State University, with the assistance of Prof. J. A. L. Waddell, on the part of the Phœnix Company, and by him accepted on the part of that company. The presumption being that it would be very difficult and expensive, if not impossible, to maintain the old line through the coming summer of 1889, everything was done with a purpose to complete the bridge by May, or, at furthest, June, of that year.

In this connection it may be said that the adoption of the cantilever was justified, even if more costly than the two spans of through trusses (which it was not, there being nearly $60 000 in its favor), the bed of the stream being such that no prudent builder would take the risk of maintaining falsework during the season of high water, into which the work must, from the lateness of the time of commencement and the dimensions of the work, probably extend. Although, for causes hereafter stated, this danger was escaped in 1889, yet in the following year the wisdom of the plan was fully vindicated, as will be shown hereinafter. Then again, when the constantly shifting channel with reference to navigation, rendered it uncertain where the channel would be during the season of low water, one of the two plans became necessary to avoid the heavy demurrage by boat owners; that is, either the two through spans or the cantilever; and the former is still open to complaint during high water as a dangerous obstruction in the rapid river. Indeed, complaint was made to the Secretary of War in April, 1890, about the time the bridge was being completed, that the present structure would obstruct the navigation of the river; and Major W. H. H. Benyaurd, of the United States Engineers, was detailed to investigate. While the report has not yet been received, Major Benyaurd expressed his conviction to be,

"that a plan better adapted to provide for unobstructed navigation could hardly be devised." To still further show the wisdom of the course taken, it may be here noted that the June rise of 1890, did carry away the falsework used for erection of the east anchor arm, although very heavy and strongly built; happily, however, after the erection had progressed beyond the point of danger therefrom.

In January, 1889, a contract was made with Sooysmith & Co., whose pneumatic plant was arriving at the site, to build the masonry and the concrete anchor piers, and to sink one caisson at Station 189 + 25. For the masonry and concrete work the prices were not changed, as, while the amount of masonry as contemplated in the original contract was very much reduced, on the other hand, the amount of concrete work was considerably increased. The caisson, 28 x 57½ feet, was built in place by the aid of the protection of an artificial breakwater, and sinking commenced. At the same time the contract for the cantilever and the viaduct approach was closed with the Phœnix Bridge Company, and arrangements made to prosecute the manufacture of the metal with the utmost speed.

The cantilever, as planned, was proportioned to conform to the grounds, the position and character of which in a great measure governed the economic location of the piers. Dividing the structure into two anchor arms of 165 feet each, and making the cantilever arms of the same length and the suspended span of even 330 feet, allowed the east anchor pier to be placed at low water level on solid rock at the east water edge; the west main pier, the same at the west water edge; the west anchor pier at the proper distance back and on a rocky ledge; and brought the east main pier on what was then deemed the best and most westerly location practicable without largely increased expense. It was generally conceded that the excessive length of 330 feet for the suspended span was not exactly the most economic; this was especially so claimed by Professor William H. Burr, Engineer of Construction for the Phœnix Bridge Company. After consideration, it was decided, for reasons above given, to adhere, however, to this arrangement, in which action Professor Burr readily concurred.

The following is given from the computations by Consulting Engineer Professor S. W. Robinson, showing that the objection to the span length as being uneconomical, had but little weight as compared to the cost of sinking to a foundation at greater depth both in and at each bank of the river.

"For the case that the center suspended truss is 330 feet or 275 feet, respectively; otherwise the same, except to be equally well proportioned for carrying the loads; we find that with the center span 275 feet, the whole bridge will weigh less than if it be 330 feet by 18.29 tons of metal, or there will be a less cost of $2 194.80 at 6 cents per pound. The principal figures run about thus:

	330 feet.	275 feet.
Weight center trusses without track or floor —tons	271.42	180.81
Difference—tons		90.62
Weight cantilever arms, not including track or floor or details common to both—tons,	402.04	447.00
Difference—tons	44.96	
Maximum moment at one main pier—foot tons	591 175	82 498.8
Per cent. of excess in anchor arms for 275 feet center		5.72
For center span of 330 or 275 feet:		
Weight of supporting members of anchor arms—tons	478.5	505.87
Difference—tons	27.37	
Total advantage for 275 feet center—tons		90.62
Total disadvantage—tons	44.96	
Difference in favor of 275 feet center = (90.62 — 72.33 = 18.29) 27.37	27.37	
	72.33	90.62."

Then again the question was raised as to the proper distance between the trusses. While at the time of making the design the same conclusion had been arrived at independently both at Abuquerque and Phœnixville, viz.: to fix this at 25 feet, center to center; and work had reached such a state of progress as to render it very expensive if not impracticable, to change the design; yet in view of the great interest in the work represented by the party raising the question, it was determined to submit the matter to some undoubted authority. This was done with no purpose on the part of the writer (at this time Chief Engineer) to change the design, but with the hope of settling the question to the satisfaction of all concerned, as to the correctness of the conclusions arrived at by the designers, originally. Indeed, with the large amount of work already done on the superstructure, and the substructure also, as well as the question of time of completion of the bridge

and the danger threatening the traffic over the line, by which the loss, direct and indirect, would amount monthly to a very large sum, the question of change could not be entertained. The question was, however, settled to the general satisfaction of all, after full consideration. It is not claimed that the design is the very best possible, but taking into consideration the haste with which it had to be prepared, and the many points to be considered, the immense amount of computation and the amount of draughting and office work, it will do credit to all engaged. The simple, yet effective manner in which the suspended span is connected to the two cantilever arms, the nicety with which all expansion and contraction is provided for, the amplitude offered for lateral and transverse bracing, and the simplicity and effectiveness of the self-sustaining arrangements during erection, are some of the points that could be cited.

The following from William H. Burr, which is pertinent to this question, may be given here with propriety, as, while the Professor is connected in a business way with the Phœnix Bridge Company, still his standing as an authority on this subject will hardly be disputed.

"Abundant lateral stability is secured in an ordinary uncontinuous truss, if the width between centers of truss is taken at $\frac{1}{18}$ the span. In fact, engineers frequently make the extreme distance between truss centers for long spans $\frac{1}{20}$ the span length; but I think $\frac{1}{18}$ is a better limit, as giving greater lateral stability. Now, by a calculation, which is needless to give here, it can be shown that if the suspended span of a cantilever opening is $\frac{4}{10}$ of that opening, then a distance between the centers of a cantilever truss of $\frac{7}{18}\frac{}{0}$ of the total opening will give essentially the same stability (lateral) as that possessed by a simple uncontinuous truss of the same length as the total cantilever opening with its truss separated by a distance equal to $\frac{1}{18}$ the span. But $\frac{7}{18}\frac{}{0}$ x 660 equal 25.7 feet, and our suspended span is one-half of the total opening, thus permitting the trusses to be a little nearer together, *i. e.*, 25 feet, instead of 25.7. Again, the chord of the cantilever and anchor arms are about four feet inside measure, making the total width of structure 29 feet, and materially enhancing its stability. For all these reasons, I have no hesitancy in saying that the Red Rock cantilever trusses are amply far apart."

The Professor might have added the further fact of the peculiar form of the bridge giving less depth of truss than usual at center instead of greater, as is the case with ordinary truss. Wind stress being the agent feared, it is proper here to say that, while the design was being made, communication was opened with Lieutenant W. A. Glass-

ford, then Superintendent of the United States Signal Service for the Pacific slope, who reported after examining the records of that department that—

"There is no record of wind so violent as to be characterized as a tornado, except in two cases, and in those the indications were that they were rather of the character of a sand gale."

At the bridge site, however, storms of this character are quite frequent, attaining a velocity of nearly 60 miles per hour, and frequently so strong as to stop work on the erection of the bridge. One occurring on the 27th of February, 1890, at which time the west half of the bridge was approaching completion, with the immense traveler on its long river arm, went very far to assure all observing the effect of the wind, that the bridge was stable. For the safety of the traveler it was lashed to the bridge, but the latter showed but a slight tremor. It had been proposed to stay the bridge with guy-lines to anchor cribs in the river, but it was found unnecessary to do so. Observation shows that the Arizona or Southern California wind storms, though severe, move with a steady flow, with scarcely a lull for hours at a time, and do not produce those sudden gusts that are found so destructive farther east. Indeed, what is known as the tornado or hurricane, is scarcely known west of the Continental Divide, or for several hundred miles eastward. While the usual provisions were made for wind stress, still there are less grounds for anxiety at that point than farther east.

Owing to the greater variation of temperature, unusual care had to be taken to provide for expansion and contraction of the metal. The lowest atmospheric temperature at that point is 26 degrees Fahr., and the highest, 125 degrees, making a range of about 100 degrees, causing a total difference in the length (660 feet) of the span between main piers of about 5¼ inches (using co-efficient .00000663 per degree Fahr.). The bridge resting entirely on the two main piers, with the suspended span supported from the cantilever arm by four large eye-bars at each end, and with the lower chord telescoping after erection wedges were removed, this expansion is provided for, half at each end. At no time under this range of temperature can any stress be exerted on the main piers. The expansion on the anchor arms is provided for by a slight rocking motion of the long anchor bars which turn on the anchor pin at the bottom of the anchor pier. The possible danger from a cold weld, should it occur on these piers, is provided for by slightly increasing the size of

these eye-bars, beyond that necessary to bear the direct stress of the bridge.

While iron lying in or upon the sand, will absorb heat by exposure to the direct rays of the sun to 165 degrees Fahr., still in the bridge structure, it will not go over 135 to 140 degrees Fahr. This will not cut any figure, as there is an excess of some (over) 4 inches in the slotted chord, fully meeting such excess. Sand with the thermometer at 120 degrees in the shade, will absorb heat to 140 to 145 degrees, while water in the river rarely exceeds 60 degrees.

Resuming the history of the work: By the 16th day of March, the work having been steadily and uninterruptedly pushed, the grading had been completed to the bridge site at both banks of the river; the Chino Quarry had been opened and most of the stone cut and conveyed to the site; the west anchor and west main pier completed and the caisson built, and the sinking of the same had progressed to the depth of 8½ feet. The metal for the anchorage had been manufactured and received, and a large portion of the metal for the bridge had been rolled and was in process of manufacture; in short, there was every assurance that the bridge would be completed in June of 1889, when an unexpected difficulty was encountered.

The caisson, at the depth then reached, uncovered a portion of the supposed bed rock, which was found to consist of a compact boulder bed. Further investigation showed that there were portions of it that could be penetrated by the drill, showing gravel and sand underneath. Work on the caisson was immediately suspended and every appliance in reach was used to determine to what depth this extended. Several holes were drilled to a depth of 64 feet or thereabouts, below low water, all indicating a thickness of 3 to 4 feet in this bed of boulders, with nearly 20 feet of what seemed to be mainly sand and gravel, and with boulders frequently encountered, but of too unstable a character to depend upon for the foundation of the pier. Below this to 64 feet was found quite compact, yet of the same general character. After repeated trials with the drills (2-inch) then used, nothing beyond the 64 feet could be penetrated; even with 3-inch casing with tools of the same size, but a foot or two more could be made. Thus it became evident that a much stronger caisson, sunk to a much greater depth, would be necessary, involving an additional expenditure of at least $100 000. The question of change of design could hardly be entertained for a moment, yet, in view of such a

large addition to the cost, it was deemed best to at once refer the matter to the management at New York and Boston.

The facts in relation to the soundings, on which the design was founded, will be given more at length hereafter, not to fix blame on any party connected with the work, but to serve as a lesson for the future. If any mistake was made beyond what might be due to the haste in which the work was done and to the unusual circumstances of the case or to the conditions there existing, the writer is not aware of it. Heavier and more perfect tools and appliances in the first place, would have gone very far to avoid the failure. These were afterwards used and the true bed rock was reached after penetrating 20 feet of large, smooth and very hard boulders, a result not often accomplished.

Meantime, work on the caisson and on the metal of the superstructure was immediately suspended, and by request of Presidents E. F. Winslow and William B. Strong, on behalf of the St. Louis and San Francisco, and Atchison, Topeka and Santa Fé Railroads, joint owners, it was arranged to have the matter investigated by A. A. Robinson, Second Vice-President, and James Dun, Chief Engineer. These gentlemen proceeded at once to examine the condition of the matter, reporting (a copy of which report is here inserted):

"ALBUQUERQUE, N. M., March 24th, 1889.

E. F. WINSLOW, President,

St. Louis and San Francisco Railroad Company.

WILLIAM B. STRONG, President,

Atchison, Topeka and Santa Fé Railroad Company.

"GENTLEMEN,—We have made a careful examination of the site of the proposed bridge across the Colorado River at the Needles, and of all the data in the office of the Chief Engineer of the Atlantic and Pacific Railroad Company, going over the estimate of cost of the various plans that have been proposed ; with the result that we, without hesitation, agreed that the best and most economical course to pursue is to make no change in the plan of crossing, except to provide a larger and stronger caisson on which to found the east pier, and sink it to the depth of 70 feet, with a possibility of having to go to a maximum depth of 80 feet below low water. We are both of the opinion that it would not be safe to sink the caisson to a less depth.

"The soundings that have been made this season show that the boulders found above this depth are in nests of very limited dimensions and that

NOTE.—Confirmatory of this deep scour, it may be mentioned that about June 1st, 1890, shortly after the new bridge was opened to traffic, one pier of the old bridge went out, throwing two spans into the river. When it is considered that the pier in question had a pile foundation to a depth of 56 to 58 feet below water, it will be seen that the possibility of scour to great depth is fully demonstrated.—*Rowe.*

they are of comparatively small size, varying from 6 to 24 inches in diameter; neither do these collections of boulders appear to be stable. We have examined the records of soundings made at this same place (point) for the Southern Pacific Company by Mr. Trainor in 1881. He was engaged for a month in making the soundings and has left full records of the same, and evidently intended to make a thorough search for the bed rock and thought he had reached it. We can only account for the wide discrepancy between the soundings made by him and those recently made, on the supposition that the bed of boulders which Mr. Trainor found and reported to be bed rock, was scoured out by the exceptional flood of 1884, and that fine sand and gravel replaced them. The comparatively small size of the boulders found in the caisson of the east pier shows conclusively that they are shifted about from one point to another at the caprice of the current, and that the fine sand and gravel in which they are imbedded, affords no reliable foundation.

"We have made as close and reliable estimates as the information at hand justifies of the cost of the three plans, which seemed best to meet the new condition of affairs.

"*First.*—A cantilever span of 830 feet, the piers of which would rest on the rock, which is exposed on both sides of the river.

"*Second.*—Two spans of 415 feet each, resting on a pier midstream (and piers on rock on each shore).

"*Third.*—To adhere to the present plan of a cantilever span of 660 feet, with the exception of sinking a larger and stronger caisson 30 x 60 feet outside dimensions, to a depth of not less than 70 feet below low water.

"The comparative cost of the three plans resulted in favor of plan No. 3, aside from damage claimed by the bridge company owing to the change of plan of the structure. This damage, the bridge company estimate, would be about $50 000, which renders the consideration of any other plan than No. 3 inadmissible. It is almost certain that the high water will occur by the 15th of May, and there is not sufficient time to procure the additional material and sink the caisson in the short interval remaining. We therefore recommend that the contract on the anchorage piers be allowed to proceed, as the stone is all quarried and nearly all cut, while the force and plant to do this part of the work are now on the ground and could not be brought back without an increase in expense. An extension of the time for completing the superstructure should be made until March 1st, 1890.

"The additional material for the caisson should be delivered in September, 1889, so that it could be framed in that month and the work of sinking be started by October 1st, 1889. We find that an agreement has been reached with Sooysmith & Co., for adjusting the value of the work performed by them to date, on the caisson which it is necessary to abandon. The sum agreed upon is $24 000, which we believe to be fair

and just. The grading for the new line is completed with the exception of one rock cut at the west approach of the bridge. About two-thirds of the pile bridging is completed, the track being carried over on temporary cribs on the remainder. The track is laid and surfaced for the entire distance, except at the site of the bridge and through the cut above mentioned. The masonry for the west pier is completed and the foundation for the anchorage piers ready for the concrete. Nearly all the stone is quarried and cut for the masonry. All the work which has been done is first-class in every respect, particularly so as regards the masonry.

"The following estimate of cost of the bridge and road when completed is submitted, which we believe will be found to be in excess of actual results, viz.:

Engineering, etc.	$25 000
Grading	84 000
Pile bridging	48 000
Steel rails	12 100
Cross-ties	13 100
Track fastenings	6 500
Track laying	12 000
Masonry and foundation of west pier completed	14 700
Loss on abandoned caisson, east pier	24 000
Cost of new caisson, east pier, and sinking same	108 000
Cost of east pier	15 600
Cost of anchorage piers	33 600
Cost of superstructure, including iron trestle approaches	219 000
Cost of bridge floor	6 500
Allowance for freight charges on Atlantic and Pacific line	15 000
Total	$637 100

"In the above estimate it is assumed that the 10 miles of steel rails and fastenings on the present line will be credited to the new line, but no allowance has been made for salvage on cross-ties and bridges on the present line between Powell and the Needles. Attention is called to the high estimate of cost of the caisson for the east pier, and we would not advise contracting it for this sum, believing that, if put down at cost and a percentage (the latter limited to $12 000), the actual cost would be at least 10 per cent. below the estimate for this item. But desiring to cover all possible contingencies, we have deemed it best to make an allowance sure to cover the same. All of which is respectfully submitted.

(Signed) James Dun,
Chief Engineer,
St. Louis and San Francisco Railroad.

(Signed) A. A. Robinson,
Chief Engineer,
Atchison, Topeka and Santa Fé Railroad."

Formal notice was sent by wire notifying the Phœnix Bridge Company to suspend work on the metal of the superstructure on the 16th of March, and on the 19th the writer received instructions, as indicated in this report, with further direction to determine the actual depth to rock at the caisson site. The contracts with the respective contracting firms having been modified to meet the changed conditions, and the first caisson broken up and removed to avoid its being buried and forming an obstacle to further progress of the work, the work was suspended so far as referred to the bridge.

It is proper to review the question of soundings here, as this experience by which the whole progress of the work was stopped and the completion of the bridge was delayed for nearly a year with the cost of maintaining the old line in the valley, impresses itself as one of the first importance. Going back to the time of making the design, while the soundings at the pier site were being made and about to be completed, the following instructions were telegraphed to Mr. Jenkins on the 25th day of December, 1888:

"Kindly arrange to have holes drilled to considerable depths, both on the site at Station 169 + 25 and the site at west edge of river 194 + 90 to see if any soft material interposes."

On December 29th, Mr. Jenkins reports as follows:

"Herewith we hand you records of soundings made at Red Rock crossing; also profile of soundings at Station 189 + 25. In making these soundings we have been careful to report only what experience has taught us is the true 'bed rock,' or the proper foundation upon which to build, but as you well know, there is always room for variation in reporting soundings where the material composing the bed of the river is so varied in character as in the present case. On the west side bottom we are positively sure that the bed rock is of the same character as that seen outcropping near the water's edge, viz.: red breccia, very compact and hard, on the east side (*i. e.*, east half of river bed—*Rowe*) the bed rock is more irregular and not so readily distinguished, but is no doubt composed of two varieties of the concrete, the softer overlaying the harder stratum. We would recommend, therefore, for a pier say, at Station 189 + 25, that the excavation be made through the upper stratum of concrete to the lower or harder material in order to obtain a proper or safe supporting power. Although but few boulders were encountered in sounding (and those small ones and principally on the east side), it is always advisable to provide a special clause in the specification for boulders, particularly if pneumatic caissons are to be used in the construction of the pier foundation."

The following is Mr. Jenkins' record of the test drill at the pier site in question, sounding No. 23 at Station 189 + 25 in accordance with the telegraphic orders just noted :

Boring No. 23 at center Station 189 + 25 :

470.64	Elevation of water surface.
3.00	Water.
467.64	Surface of sand.
6.19	
461.45	Gravel (or drift), boulders and sand.
0.61	
460.84	Surface of breccia, red in color, hard and compact,
5.63	but easily broken up with drill.
455.21	Hard stratum of solid substance, probably "breccia" of same character as eastern shore.
9.80	to bed rock—15.43 to harder stratum.

The tools used by Mr. Jenkins consisted of a 1¼-inch drill, or what is termed the "jet drill," working in a 1½-inch casing, for which ordinary wrought-iron pipe is often used. Steel pipe, however, was provided as being stronger. The drill rod was made of three-quarter-inch pipe, and the jet was furnished by a double lever hand force pump. This outfit being light, was well adapted to use in the river by aid of a light flat-boat, anchored at the desired spot, and was found to penetrate compact gravel readily, even passing directly through boulders of 1½ feet diameter. Usually, however, it would either shift or turn the rock aside, or, owing to the elasticity of the casing, deflect to the most yielding side and so pass on.

Immediately on the discovery of the difficulty at the caisson site, heavier tools were made, 2 and 3-inch, and pending the decision as to what course would be decided upon, several holes were pushed down in and about the site. It was found that, while difficult to penetrate the first bed of boulders which were laid bare in the caisson, yet some hours of persistent labor would accomplish this, and the drill would proceed with little further difficulty until the depth of 64 feet was reached (elevation 406.00), when in every case the work would stop, either from breakage of the tools by long and persistent pounding or by the collapse of the casing.

In June following, heavier tools of much the same character were pro-

cured and placed upon a flat-boat; a No. 6 steam force pump, and an engine to run the machinery, were provided. The 3-inch tools were first used to the depth of 64 feet, before reached. The casing used was strong 3-inch wrought iron pipe, armed with a steel shoe to strengthen the end coming in contact with the work. Beyond this, however, progress was very slow and difficult; boulders were encountered at every move, and it is presumed, were in most cases cut through or broken, as they were imbedded in compact gravel cemented into a mass by a silicious cement, preventing any movement to allow the casing to pass. When the casing could no longer be driven, the 3-inch tools would be laid aside and a 2-inch casing and drill would be introduced inside of the 3-inch casing. This would be driven forward a foot or two until either the tools or the 2-inch casing would collapse; then withdrawing the 2-inch casing and tools, the 3-inch would be again resorted to. Thus foot by foot would be gained. The 2-inch casing, when withdrawn, would somewhat resemble the letter "S," becoming so distorted as to prevent the movement of the drill rod in it. During the whole progress from the first sand bed to the boulder bed, the jet would throw out a constant stream of clear, river washed sand, from which the cuttings of the drill were difficult to distinguish.

When, however, the boulder bed was entered, the cuttings became more plentiful, and the character of each stone passed through was as well indicated, almost, as if the boulder could be seen. Pure quartz, gneiss, porphyry, trap and the various types of volcanic rock and some very fair specimens of natural concrete were among the exposures (note at the time that this formation was afterward reached in sinking the caisson, bits of partly siliciureted wood were found above, on or near the boulder bed, which would indicate somewhat the great length of time this formation has occupied its present position).

Finally, on reaching 80.7 feet in depth, the sand thrown by the jet changed, or rather became highly colored and reduced in quantity, and then finally ceased, and well identified cuttings of the red breccia were obtained, identical with that outcropping on either shore. Thus, the first test hole was located at the 189 + 08, and 18 feet south of the center line of the bridge. Another was put down at the 189 + 50, about the same distance from the center line on the opposite side, of which it is unnecessary to say anything except that the same formation was reached, at $\frac{8}{10}$ foot less depth; otherwise the history does not materially differ.

In each case the bed rock was penetrated about 1 foot. In both the casing was so contorted by its encounter with large boulders, as to prevent further progress without undue expense. Indeed, the formation was so thoroughly identified as to render it unnecessary, especially as the character of the material below the elevation 406, as developed by all the borings, forced the conviction that it would be very difficult, if not impossible, to sink a caisson below that point. There are the best of reasons for believing that a caisson sunk half the distance, or to the elevation of 436, would have been entirely secure, as from that elevation, the boulders were found so numerous and so compactly embedded in cemented gravel, that it was highly improbable that the river would ever, in the lifetime of the bridge, come to that depth at that point.

The lesson taught may be of value in the future and in this connection every hint should be heeded and every event scanned. The fact mentioned by Mr. Jenkins as to the change of character in what was supposed to be bed rock, from that well identified on the west half of river bed and that found on the eastward, it can be seen should have cast suspicion on the conclusion drawn, as well as the very moderate depth, at a point so distant from the east shore. Again, the conformation of the river, by close examination of the vicinity of the bridge, as well as the character of its drift, should have indicated the probable presence of the accumulation of drift; while the conformation for a mile or two above would have sufficiently indicated its ancient character. By examining the map of the bridge site, it will be readily understood that the tendency of the channel is westward at that point, while the rock bound bank on the west side of river for a mile or more above, with the further influence of a large cañon entering the river above the head of the Needles Cañon on the same side, with its frequent discharge of drift including boulders of considerable size, tend to crowd the river channel eastward. The deep scour of the river must occur at time of extreme flood, at which time its course is most in conformity with that indicated on the map, and it is hardly possible that it should attack the pier site.

The soundings in the Colorado River were unusually difficult, both in consequence of the great depth, and the heavy, coarse character of the drift, to which were to be added the evanescent character of its quicksands and the violence of its current. From all those combined with

the haste in which the work was prosecuted, it is only surprising that more mistakes did not result. Had the suggestion made at the time of making the preliminary report been followed, time and money would have been saved, as well as much embarrassment. The engineer must know what the conditions are before he can either plan or proceed with the erection of a structure of this kind with any degree of safety or credit. In any similar case larger and heavier appliances still than those last used would be safer, particularly when boulder beds are apprehended, as "well men" all know the fact that a formation of that character is the most difficult of all to penetrate.

For the benefit of others in the future, it is here stated that it was found that a water pressure of 18 to 20 pounds on the jet was the best; a heavier pressure would raise gravel and fragments of rock to such extent as to impede the free stroke of the drill rod. With softer rock, perhaps a less pressure would be necessary. The sands as well as the rocks in the Colorado River are nearly 15 per cent. greater in specific gravity than those of other localities farther east. And further it may be said, that in work of this character, extreme care must be taken to not overstrain the tools, thus incurring breakage or disconnection of the drill rod coupling. It was found that by changing drills and reconnecting the rods every five hours, work could be safely and steadily prosecuted, while any disregard of this precaution would result in a job of fishing, the nature of which every experienced drill man will appreciate.

The result as above having been attained in July, 1889, the material for the caisson was ordered and received, so that in the latter half of September, the work was again resumed; the tramway (Plate CXVII) by which the caisson site was reached was partially rebuilt, about 200 feet having been swept out by the high water of June; 20 feet depth of the river bed having been swept out and again replaced, as the river subsided. On November 21st, air was put onto the caisson and kept on continuously until February 11th, when the filling of the caisson was completed, resting on the boulder bed at elevation of 409.00. The sinking progressed steadily from the first, and the corners did not get out of place to exceed 4 or 5 inches either longitudinally or laterally, during the whole progress of sinking. The material proved even harder and more firm in character than was inferred from the boring. Owing to the unyielding character of the material surrounding the caisson, comparatively a small

PLATE CXVII.
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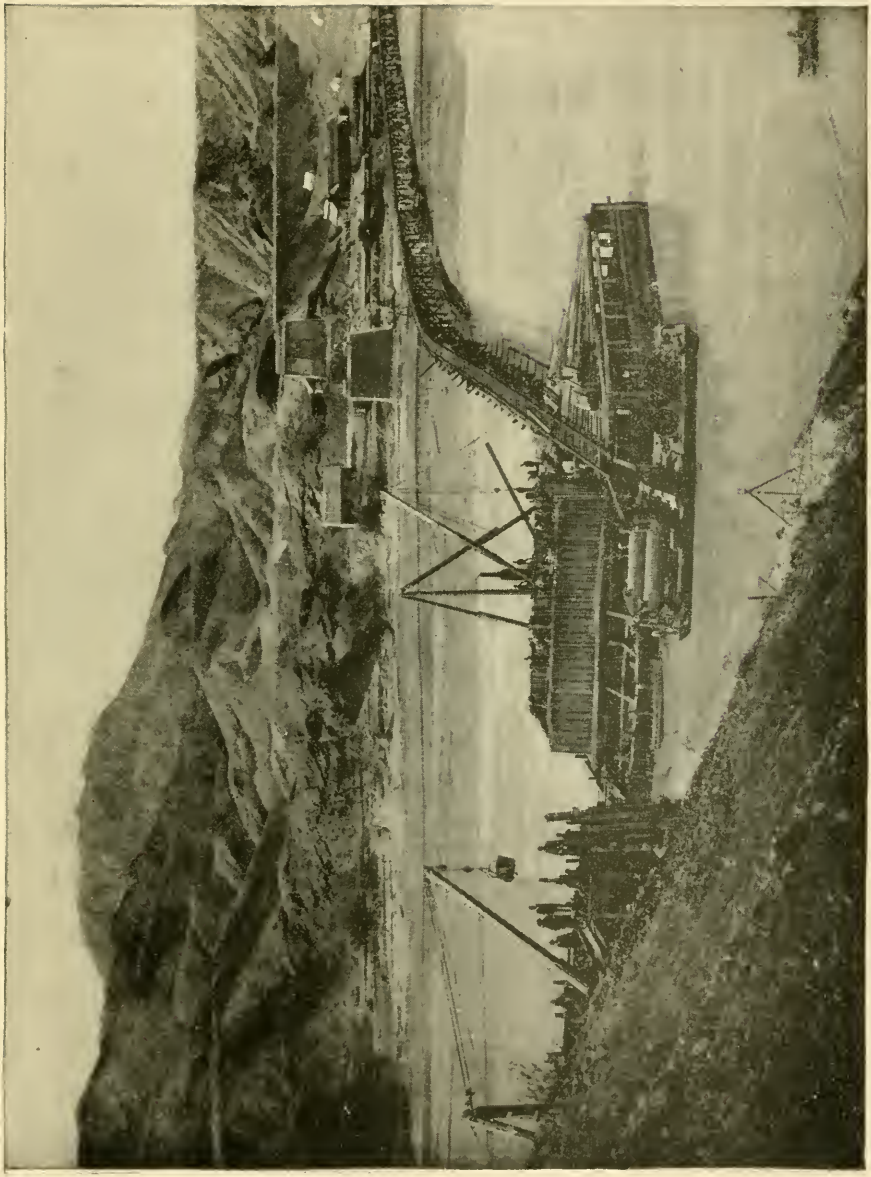
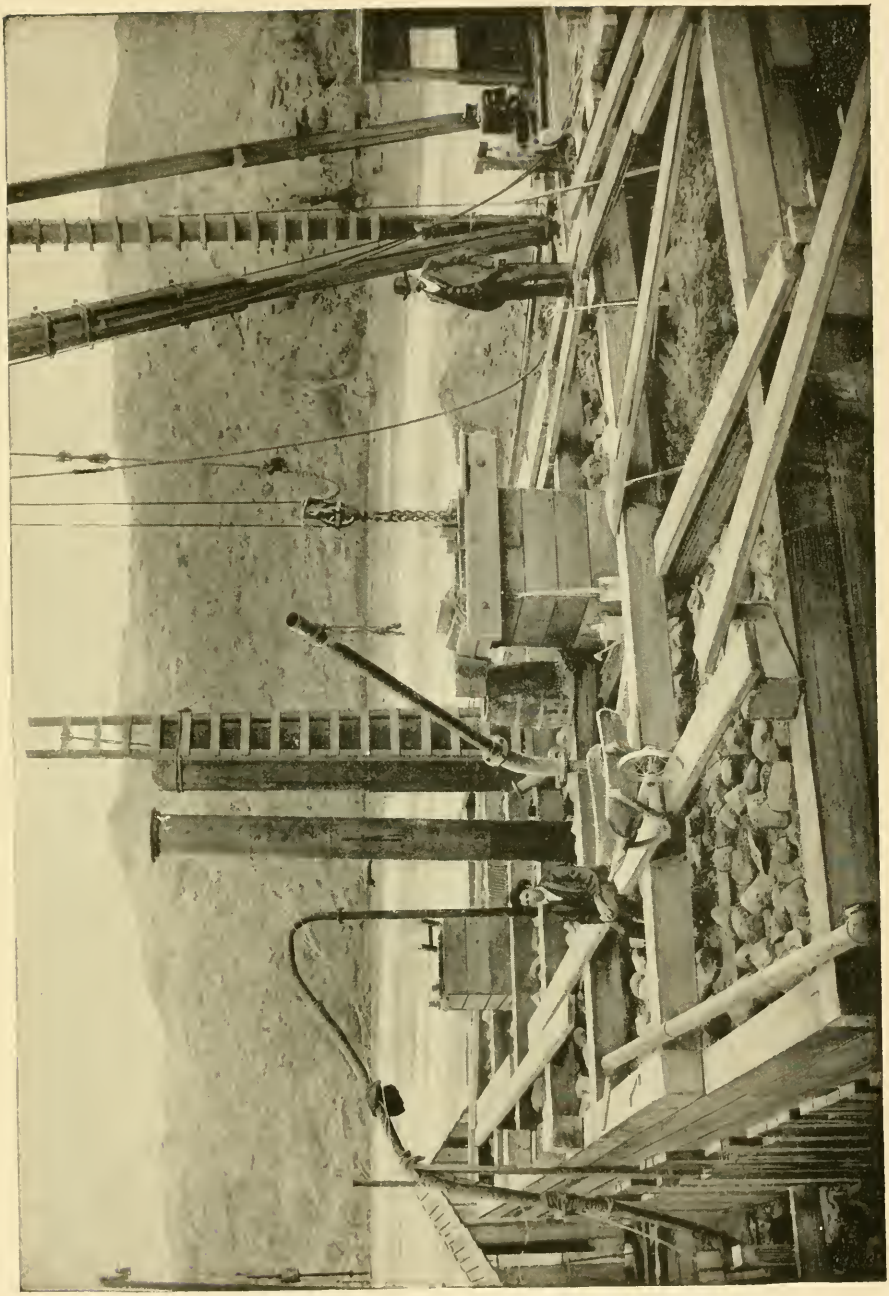
RED ROCK CANTILEVER BRIDGE.





amount of excavation was made beyond its area. The caisson, a plan of which is shown in the photograph (Plate CXVIII), was carefully and strongly built, and measurements made at the time of sealing, showed that it had borne the strain with scarcely any distortion. The table appended gives a history of the progress in sinking.

To determine in advance just when to stop the crib and commence the footing courses of the pier, was very difficult. In order to reach elevation 406 required that a portion of the pier should be built, as the great distance through which the caisson had already sunk through rough, unyielding material caused great friction on the sides of the crib, and made it necessary to withdraw almost or quite all the air pressure to move it. It was decided to stop the crib at a height that would put it about 4 feet below low water, so that the timber would not be exposed; but at the same time, to crowd the caisson down to 406 or possibly 2 feet more to 404. To do this, it was found necessary to put on all the footing courses and about one-half of the neat pier, and afterward lay temporarily two additional courses. Notwithstanding this, the caisson failed to settle beyond 409. After repeated trials to cause it to do so, and after making a thorough examination of the foundation, it was decided to excavate from 2 to 2½ feet all around under the cutting edge, which could readily be done, the wall standing perfectly firm, and follow with the beton in sacks until it reached the cutting edge at every point, which was done. The center was then excavated and sealed, followed by filling the working chamber. The caisson was already about 5 feet into the boulder formation at the northeast, 3 feet at the northwest, 2 feet on the southeast, and down to it on the southwest corner, indicating that its surface sloped to the southwest. The excavation made and replaced with "beton" fully reached the firm foundation at all points, and over the whole extent of the chamber. The boulders formed the greater mass of the underlying stratum and were bedded compactly, so that in excavating, each separate stone had to be picked or pried loose. While the chamber was being sealed there was a steady daily subsidence of the caisson of about .05 feet, but when the beton reached the cutting edge, this ceased and no further settling occurred up to the time of completion of the bridge. This movement was utilized to right a slight tilt of the pier to the west by sealing that side first. The slight remaining tilt was taken out in the subsequent courses of masonry and the pier finished systematically, although slightly larger

than designed. It might have been possible to spend a larger sum of money in trying to force the caisson lower, but the author doubts if it was possible to do so, and no good end would have been subserved. The top of the crib is about at the level of extreme low water.

Of the material used, each part will be treated in its turn. A record of the progress of the work of sinking the caisson is given in Appendix "A." The timber used for the caisson and crib consisted mainly of what was termed Oregon pine (yellow fir), a most excellent timber, strong and firm, though somewhat coarse grained, which weighed 35 to 40 pounds per cubic foot when well dried. The amount used in the different parts was about as follows:

	Cubic Feet.	Feet, Board Measure.
Working chamber, including 3 inches inside casing	6 880	= 82 560
Roof, 8 feet thick	12 992	= 155 904
Crib, including 3 inches casing outside	20 071.5	= 240 855
Making total amount of timber (neat)	39 943.5	= 479 319

Iron bolts, stay and drift and steel spikes 29 tons = 58 000 pounds.

Concrete (beton) put into crib, 47.7 cubic yards per running foot	2 290	cubic yards.
Concrete (beton) put into working chamber	580	"
Total	2 870	"

weight at 4 050 pounds per cubic yard, saturated.

Timber at 35 pounds will absorb water to nearly 80 per cent., so that the timber at 35 pounds when dry, and absorbing 80 per cent. (= 8 x 35 = 28), will weigh 35 + 28 = 63 pounds per cubic foot, or have about the same weight as water. Then taking the caisson at the time it stopped we have, 10 800 square feet of surface on which the pressure of the material outside tended to produce friction; the excess of the weight of the concrete in the crib over the displacement of the water, which, being 101 203 cubic feet, at 62½ pounds per cubic foot, weighed 3 162 tons. At this time there were also 125 cubic yards of masonry on the crib, equal to about 800 tons. Therefore, we have—

2 290 cubic yards of beton in crib =	4 637.25	net tons.
125 cubic yards of masonry	804.00	"
Total	5 441.25	"
Resisting this is the uplift of the water	3 162.00	"
Leaving the downward pressure of	2 279.25	"

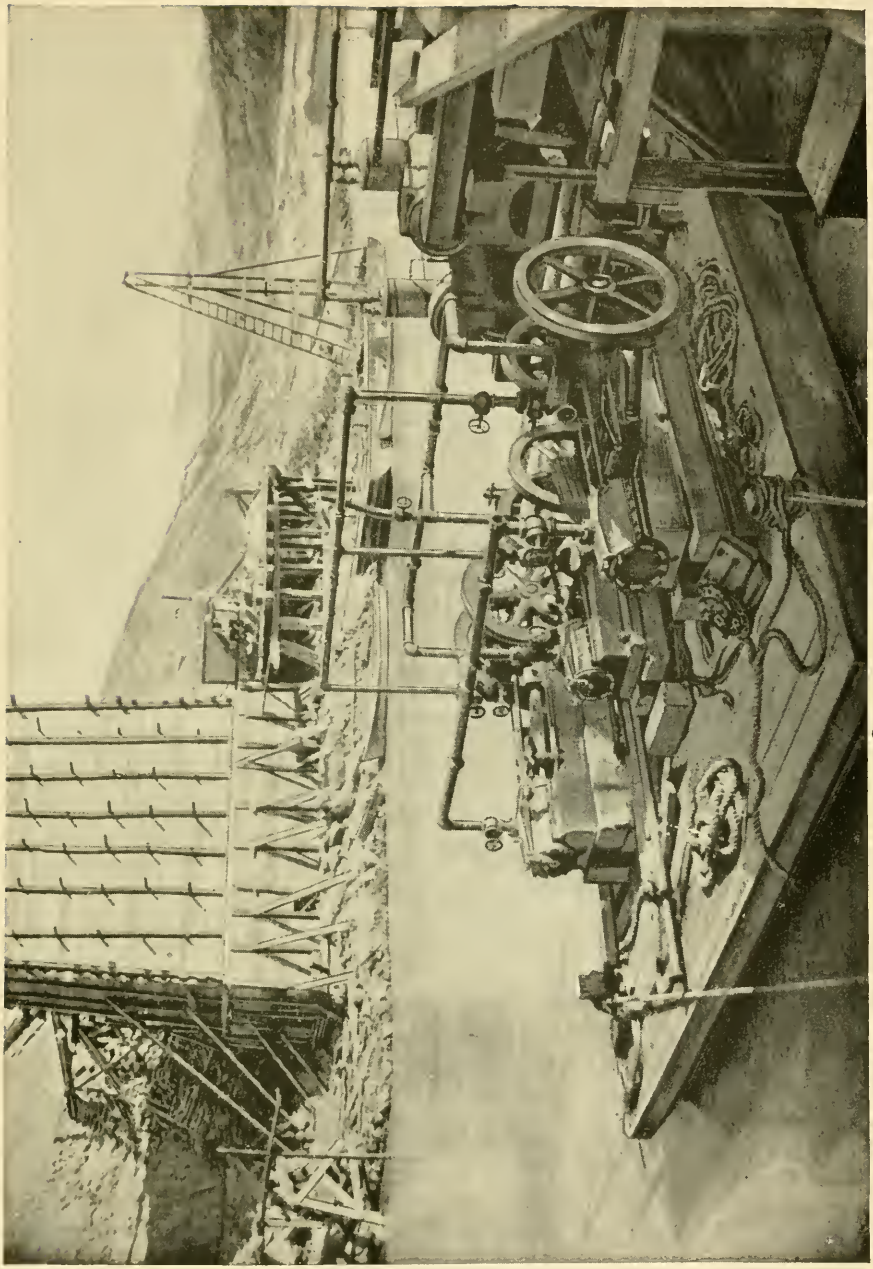


PLATE CXX.
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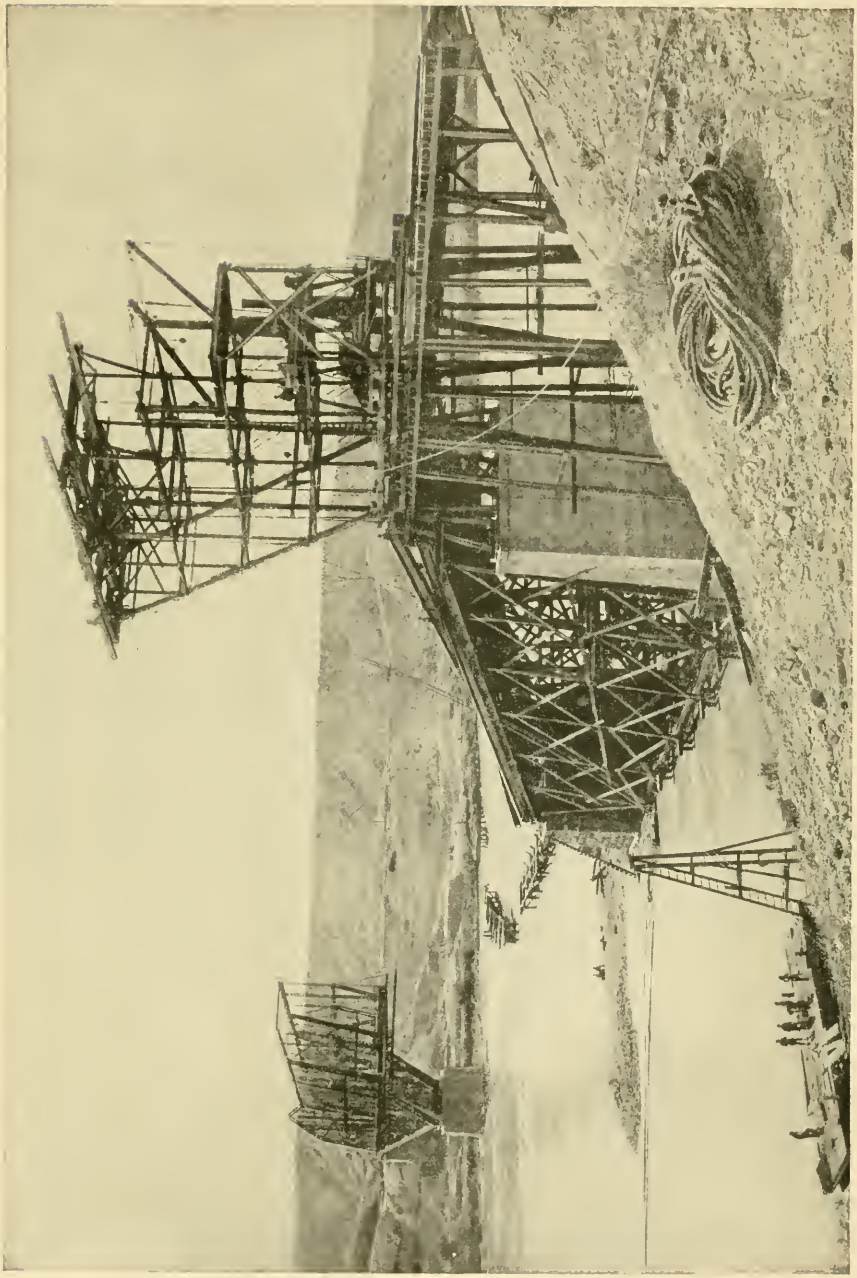
RED ROCK CANTILEVER BRIDGE.



equal to about 420 pounds per square foot when the air pressure is entirely off. When the air was on at 27 pounds per square inch, this tendency downward was entirely overcome. The air was entirely and repeatedly removed toward the close, and much of the surrounding material drawn. In ordinary sand and gravel the caisson could have been pushed to any depth desired. It is presumed that boulders were crowding between the pier walls of the excavation and the sides of the crib to such extent as to sustain it, notwithstanding the sudden release of the air, causing the sudden application of the 2 000 tons as before stated.

The whole resistance was below the elevation of 440.00, where the material was all of hard, firm gravel and boulders. The following will give much information as to the relative cost of sinking, taking labor roll alone:

November, 10 days, total roll.....	$2 611 72	Depth made 9.94 feet, mean cost per day $261.18, per vertical foot.....	$262 75
December, 31 days, total roll.....	10 026 05	Depth made 23.38 feet, mean cost per day $323.42, per vertical foot....	428 83
January, 31 days, total roll.....	10 710 50	Depth made 26.80 feet, mean cost per day $346.83, per vertical foot.....	*400 00
February, 11 days, total roll.....	3,759 05	Filling chamber 11.5 feet, mean cost per day $341, per vertical foot...	326 00

The less amount of cost per foot for January was owing to the use of the air pressure for raising much of the material during that month. The pressure used reached 27 pounds, which was maintained during the time of filling the working chamber. The air plant consisted of three compressors, two of which were double cylinders 16 x 24 inches, and one 12 x 18 inches. Two were used while excavating and one held in reserve. These were driven by two 75 horse-power boilers and one of 50 horse-power. The whole air-plant was placed on a boat 24 x 60 feet, built for the purpose (Plate CXIX), and all housed in to protect it from the sand storms. Fuel (coal) was unloaded from the car in reach of the compressor. Water was drawn from the river, and settled in a submerged flat boat as far as possible, but was always bad, being destructive to the boiler flues and causing much trouble. The 25th day of February saw the substructure complete and the west half of the superstructure nearing the center point of the bridge (Plate CXX).

The stone used in the piers, coping and bridge seats was obtained at the Chino quarry, about 140 miles east of the bridge site, in Arizona,

*The cost of the last foot or two of sinking was nearly $2 500 per foot.

where there is an extensive ledge. The color of this stone is a light, clear, cherry color, free from stain, and, laying as it does in thick ledges, it can be quarried of almost any size and shape desired. It is a compact sandstone, very hard and strong, homogeneous in texture, weighing about 160 pounds per cubic foot (4 320 pounds per cubic yard), and cutting a little easier than granite. A block of the stone 3 x 3 x 6 inches was tried by Prof. Waddell, at Kansas City, and remained unbroken after being subjected to 12 000 pounds per square inch (the extent of the capacity of the press). In the quarry the courses run from 20 inches to 6 feet, but owing to its homogeneity the stone can be split into any dimension or shapes desired. In constructing the two main piers many of the headers overlapped at the center of the pier where the pier was 15 feet thick, and some of the stretchers show 9 feet in length on the face of the pier. The courses varied from 26 down to 18 inches in thickness, with coping 22 inches. The main seats for the four pedestals carrying the whole weight of the bridge are 9 feet square and 2½ feet thick, made up, however, of four sections each. It was first designed to use stones of full size, but in consideration of the excessive cost and difficulty of handling and placing, without much heavier appliances than were at hand, it was decided to use the latter.

	Pounds.	
The anchor arm, including floor, weighs	593 900	Then as there are two seats on each pier $\frac{3\,943\,550}{2}$ = 1 971 775 pounds on each seat. Each seat 7 x 7 feet = 7 056 square inches, and $\frac{1\,476\,775}{7\,056}$ pounds = 279.4 pounds per square inch, or a little over $\frac{1}{6\frac{1}{0}}$ of the ultimate crushing load.
The cantilever arm, including floor, weighs	613 100	
The two bridge seats, pedestals and piers	79 850	
The suspended span, one-half including reaction of anchor	656 700	
Live load on all at 3 000 pounds per lineal foot	1 980 000	
Making a total weight in pounds.	3 943 550	

The two main piers (Plate CXXI) and the abutment and pedestals supporting the viaduct were built of this sandstone, as well as the coping of the two anchor piers.

The anchor piers (Plate CXXI) were built of beton, the specifications for which will be found in the Appendix E. The west anchor pier was built with hand mixed beton, as well as the foundation for the east anchor

pier and the west main pier. At the resumption of work in October, 1889, however, a mixing machine run by steam and having a capacity of 150 cubic yards in ten hours, was used, not only economizing in labor, but giving better work. The Gillingham English Portland cement was used and the specifications varied somewhat. The concrete for the east anchor pier, caisson crib and working chamber were provided with a composition as follows: The mixer being arranged with carriers to measure each ingredient, was arranged thus: Four cups cement, thirteen cups sand, and sixteen cups broken stone, carried up and discharged into the trough of the machine. A minimum of water was discharged into the sand and cement so that they became mixed before receiving the broken stone. The water was so gauged that it should not appear when the concrete was placed in the work and well tamped. The concrete was discharged into wheelbarrows or one-third yard iron buckets and conveyed to the work. Meantime, a quantity of one-man stone was distributed over the work to an amount of perhaps one-half of the amount of the concrete, and the latter driven around, between and over the stone. It was required that the broken stone should be well wetted before feeding into the machine, as well as the stone used in the work, so that thorough contact with the mortar should be secured. It was also required that each course in the pier should be well wetted after the tampers had gone over each portion of the work, to the end that all crumbs of mortar or dust should be smoothed down, and a clean surface should be secured, to give more perfect bond to the subsequent course. To prevent possibility of the feed of cement being interrupted, an inspector was placed at the machine to guard against this as well as to govern the amount of water used.

So important is this, and the frequency that it will occur, that it would be well to so arrange the feeding hopper, especially of the cement, that it would feed itself from a hopper holding a considerable quantity. The importance of having the proper amount of water and no more, will be understood when it is considered that to the packing quality of the concrete, its superior strength as a beton is due, as well as its less tendency to shrink. It is well known that, if well packed, perfect adhesion to the stone with which it comes in contact will be secured. If imperfectly packed from having too much water, it will shrink and break its bond. In the main piers the same concrete mixture with a slightly larger allowance of cement, omitting the "one-man" stone, was used.

First laying the face stones of the course and filling and ramming the face seams with mortar; the backing was all so laid as to leave room for ramming between; then the proper quantity of concrete was mixed and it was so deposited as to secure thorough packing, in several courses, until the whole course was leveled up; not forgetting to first wet all the stone with a sprinkling pot. It is not doubted that the pier will have all the stability of a monolith; the course pursued is much preferable to the practice of using drowned mortar and spawls, and the concrete will be less likely to separate than even well jointed solid courses.

The cement, of which nearly six thousand barrels were used, was tested by taking numerous briquettes from the quantities mixed by hand, and by the machine, and were found so uniformly good that the only precaution used was to break the briquettes from several barrels together, thus avoiding the bad effect from an occasional barrel that might fall below the standard. At a time before commencing the work, enough tests of neat cement were made to indicate a surprising uniformity in this brand. The test was usually made on ten or twelve briquettes each day, taken from the machine and allowed to dry in a shaded place twenty-four hours; then immersed in water for the same period, and then immediately broken. The testing machine was the "Fairbanks," and the briquettes gave a tensile strength from 160 (exceptional) to 225 pounds per square inch, a mean of nearly 220 pounds. Samples kept thirty days gave over twice that amount (410 pounds mean); some gave over 500 pounds. It was early found that the strength was governed largely by the character of the sand. No drift (wind) sand was used, as it was found to give very low tensile strength, but a bed of clean, water-washed sand was found convenient, having quantities of fine gravel intermixed, and later, a large bed of coarse sand in the bluff east of the river, so well adapted to the work as to justify its shipment around nearly 30 miles, to use in the caisson. Particular attention is called to this sand as worthy of shipment a long distance, for public buildings and other important work. A comparison of tests shows that this sand gives 30 per cent. excess of strength over good river sand (525 pounds at thirty days).

The broken stone was at first supplied from the débris of the Chino Quarry and from the volcanic rock found in the vicinity of the bridge, but it was found that the broken volcanic rock with which the "mesas" were strewn, could be collected at less cost, and being of the

same character, was substituted in the caisson work at a saving of nearly $1 per cubic yard. The process of gathering was to rake these fragments of stone into windrows and haul them by wagon to a pile where convenient to load into a car when needed. An inclined screen was erected to separate the dust from the stone while conveying it to the car. Indian labor was used very successfully for this as well as for labor about the caisson.

The criticism has been made that the anchor piers should have been made with masonry rather than concrete. In reply, it is answered that, while making the original design, the concrete was considered most suitable and cheapest. Masonry of the character required would have cost $20 per cubic yard, excluding cement, owing to the complicated structure, it having twelve angles instead of four as in an ordinary solid pier. The comparison would run thus:

Masonry, per cubic yard	$20 00	
One-half barrel cement	2 00	
Freight on stone, per cubic yard	3 00	
Making total	$25 00	per cubic yard.
Concrete, contract price	$12 00	
One and a quarter barrels cement	5 00	
Making total	17 00	" "
Making a difference of	$8 00	" " in favor

of the concrete, and (there being in the two anchor piers 1 770 cubic yards) a saving of $14 160. Taking into consideration its better adaptation to the purpose, the criticism is answered. These, at least, are the considerations which impelled its adoption.

The anchor piers have not less than 700 cubic yards of concrete over the anchorage, which at 4 050 pounds per cubic yard, will give a weight on the anchorage of 2 835 000 pounds.

The uplift will not exceed the following:

Reaction of excess of weight of cantilever over anchor arm	19 200	pounds.
" " half of dead load of suspended span	328 350	"
" " live load on cantilever arm $\frac{165 \text{ feet} \times 3\,000 \text{ pounds}}{2}$	247 500	"
" " live load on suspended span $\frac{330 \text{ feet} \times 3\,000 \text{ lbs.}}{2}$	495 000	"
		1 090 050 pounds.

(The heaviest train obtainable so far, that made up of coal, does not exceed 2 000 pounds per linear foot.) This does not take into consideration the cohesion of the concrete, which is greatly in excess of the simple weight of the mass. It would not do, however, to rely upon this altogether in any case, as notwithstanding the utmost care, the strength at the joints between courses will be less strong than the body of each course. As the anchor metal is so interwoven with the mass of the base of the pier, the above weight must be ample.

Location of Piers.—As a supported tape measurement across the river was impracticable, the pier centers had to be located by triangulation. A base line 850 feet long on the west bank of the river was carefully measured, and from that two permanent monuments were fixed on the center line, one being on each bank sufficiently distant to remain undisturbed ("*A*" and "*C*," Plate I). This line was measured with two 10 feet iron poles furnished by the Phœnix Bridge Company, and correct at 70 degrees Fahr. temperature. Later a tape was drawn across, supported partly by the tramway and the balance by the suspended stretcher 100 feet long, confirming the triangulation within $\frac{4}{100}$ foot. Then again, during the suspension of work in the summer of 1889, a large sand bar formed on the west side of the river, rendering it practicable to secure a good base line perpendicular to the center line of the bridge from the west main pier, which was done. Much care was taken and repeated measurements made at different temperatures, and they were found after applying the correction (.00000663 per degree Fahr.) to agree within $\frac{1}{100}$ foot in 702 feet, the base being extended so as to use a parallel base 42 feet from the center line of the bridge. From this an angle of 45 degrees was carefully and repeatedly turned and reference points fixed at such a distance as allowed a clear sight over the east pier when finished. By these reference points the bridge pedestals were set on the east main pier, and the bridge erected. When the bridge erection met at the center of the river, the lower chord bars overlapped $5\frac{5}{8}$ inches, and pins were inserted and quickly driven, as soon as the lower erection wedges were slightly slacked off.

Completion of Bridge.—The bridge was completed on the 25th of June, 1890, in a manner creditable to the Phœnix Bridge Company, the contractors, every part going together without a hitch and with commendable speed. It was, however, opened for the passage of trains on the 10th of May, by the removal of one engine and the floor

of the traveler, this being necessitated by the long expected disaster to the old line, in spite of strenuous efforts to maintain it by the Road Department. Within two weeks, too, the old bridge lost a pier and two spans.

The shop inspection was placed in the hands of Hildreth & Nettleton, and owing to the dissolution of that firm was carried through by R. W. Hildreth & Co. Later, the senior member, R. W. Hildreth, acted as Inspector of Erection. To avoid useless repetition, the report of R. W. Hildreth & Co. is given partially as follows:

(Copy.)

"Inspection of Red Rock Bridge.

"Material:

"The material used in the manufacture of the superstructure of the Red Rock Bridge was wrought-iron, with the exception of the working section of the main truss members, which was of open hearth steel. The viaduct approach was entirely of wrought-iron.

"The material purchased by the Phœnix Bridge Company of the following rolling mills: Phœnix Iron Company, Phœnixville, Pa.

"All angles, squares, rounds and flats not over 12 inches wide, Pottstown Iron Company, Pottstown, Pa.

"All wrought-iron plates over 12 inches wide, Charles Huston & Sons, Coatsville, Pa.

"Steel filled iron plates for anchorage girders, Pennsylvania Steel Company, Steelton, Pa.

"Open hearth steel blooms to be rolled by Phœnix Iron Company into shapes and flats under 12 inches wide, Midvale Steel Company, Nicetown, Philadelphia, Pa.

"Steel pins, Carbon Iron Company, Pittsburgh, Pa.

"Open hearth steel plates over 12 inches wide and steel flats for eye-bars.

"Rolling was started at Phœnixville and Pottstown in the latter part of January, 1889, and continued intermittingly at the various mills until March, 1890.

"The requirements for steel being for three-quarter round test specimens, rolled from special ingots cast with each heat, are for the purpose of insuring the proper quality of metal. A number of tests from finished steel were required by the specifications, to serve as a check of the heating and rolling. The requirements for both iron and steel, while necessitating good material, were easily met by the rolling mills; we were, therefore, obliged to make very few condemnations on the results of tests.

"At the Carbon Iron Company we were obliged to take all tests from the finished plates and bars, as these mills had no facilities for rolling rounds. This was preferable, however, as it not only gave the quality of the material as well as the three-quarter inch round would have done, but also showed whether or not the steel had been injured in the subsequent process, as of heating and rolling, more thoroughly than the specified number as taken on shapes, etc.

"For every specimen of steel taken for a tension test, a similar one was taken for bending. In all cases these specimens were successfully bent, cold, 180 degrees to close contact. Considerable material was

condemned at every rolling mill; this was rarely, however, for poor quality, but on account of surface defects and flaws.

"Tests of full-sized eye-bars were all made on the 600-ton hydraulic testing machine at the Athens shop of the Union Bridge Company. Only two of this series of tests broke in the head, and, in each case, this was due to the flaws discovered during inspection, the bars being selected for tests on that account. The result of the test of the 5 x $\frac{15}{16}$-inch bar was good, considering that the fracture was in the neck and caused by a flaw, so no retest was ordered. The retest of the 8 x $1\frac{15}{16}$-inch bar gave good results for a bar that had already been strained so much beyond its elastic limit. These tests not only showed that the proportions of the heads were sufficient to break the body of the bar in all cases unless weakened by flaws, but that the material of the bars had not been injured by the manufacturing process to which it had been subjected at the bridge shop." (See Appendix "F.")

There was no formal test of the bridge after its completion owing to the difficulty of securing engines enough to produce the typical strain. As a matter of interest to all engineers, this should still be done, if practicable. Two 91-ton engines and a train of coal cars fully loaded (Plate CXXII), sufficient to cover the bridge from one main pier to the other, estimated to be about 66 per cent. of a full test load, gave a total depression at the center of $3\frac{3}{8}$ inches.

In concluding this report the writer desires to give credit to all those connected with this important work. Mr. C. W. Smith, General Manager; A. A. Robinson, Chief Engineer on the part of the Atchison, Topeka and Santa Fé Railroad Company, and James Dun, Chief Engineer on the part of the St. Louis and San Francisco Railroad Company, advisory engineers, throughout its construction; Prof. S. W. Robinson, Professor of Mechanical Engineering, Ohio State University, Consulting Engineer; Martin Rapp, and W. F. Behrens, Assistant Engineers; R. W. Hildreth, shop and field inspector; each and all have the thanks of the writer for their efficient aid. The designers, Prof. William H. Burr and Prof. J. A. L. Waddell, the Phœnix Bridge Company, and Sooysmith & Co., contractors, each should share the credit of a good job well done. The accompanying views were secured by the aid of F. E. Evans, of Oakland, California.

PLATE CXI.
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PLATE CXI
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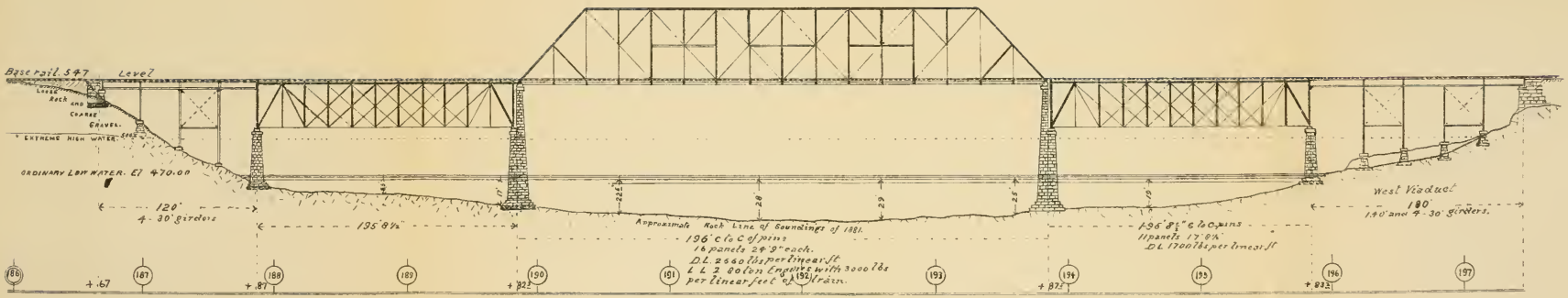
VOL XXV No 518
RED ROCK CANTILEVER BRIDGE

KEYSTONE Bt Co, Design, Modified.

RED ROCK CROSSING OF THE COLORADO RIVER.



Sept 5th 1883

PLATE CXIII.
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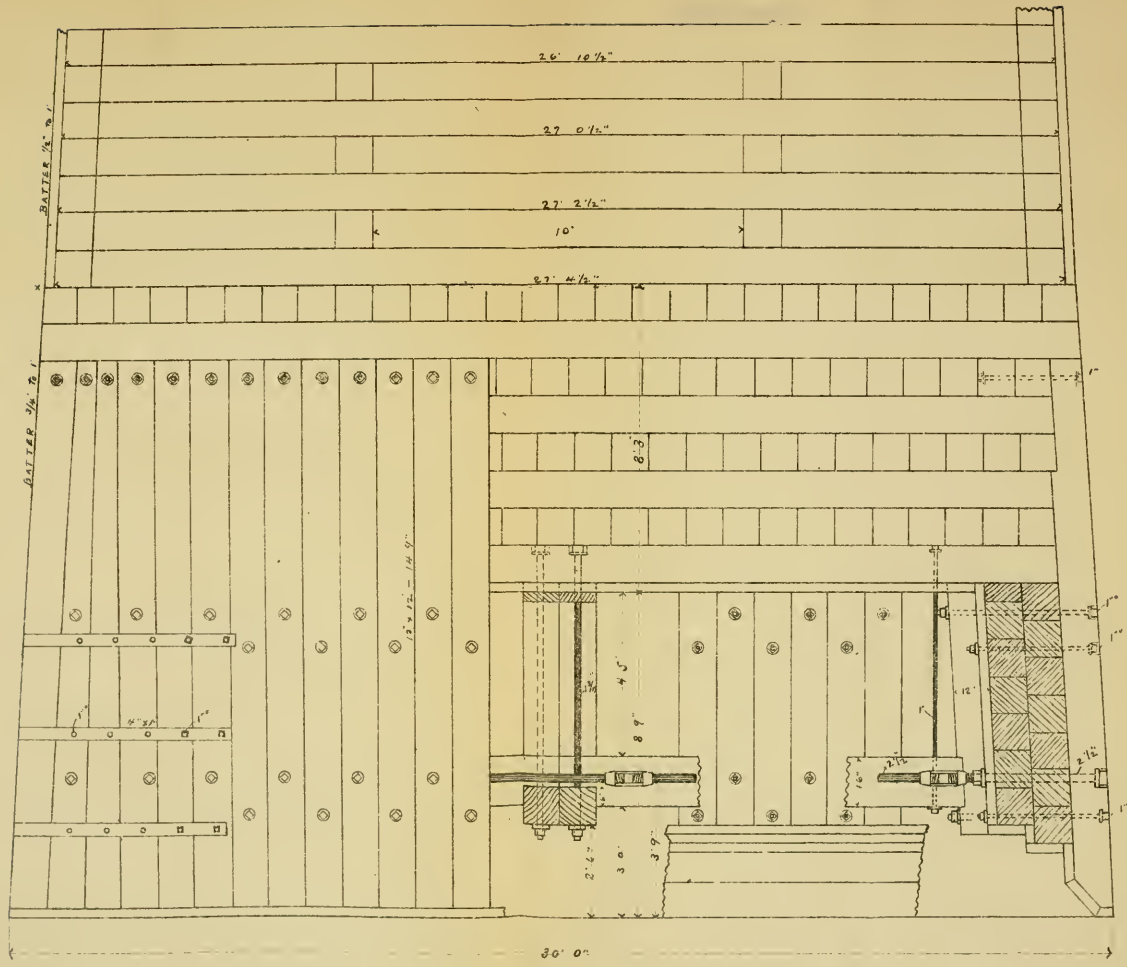
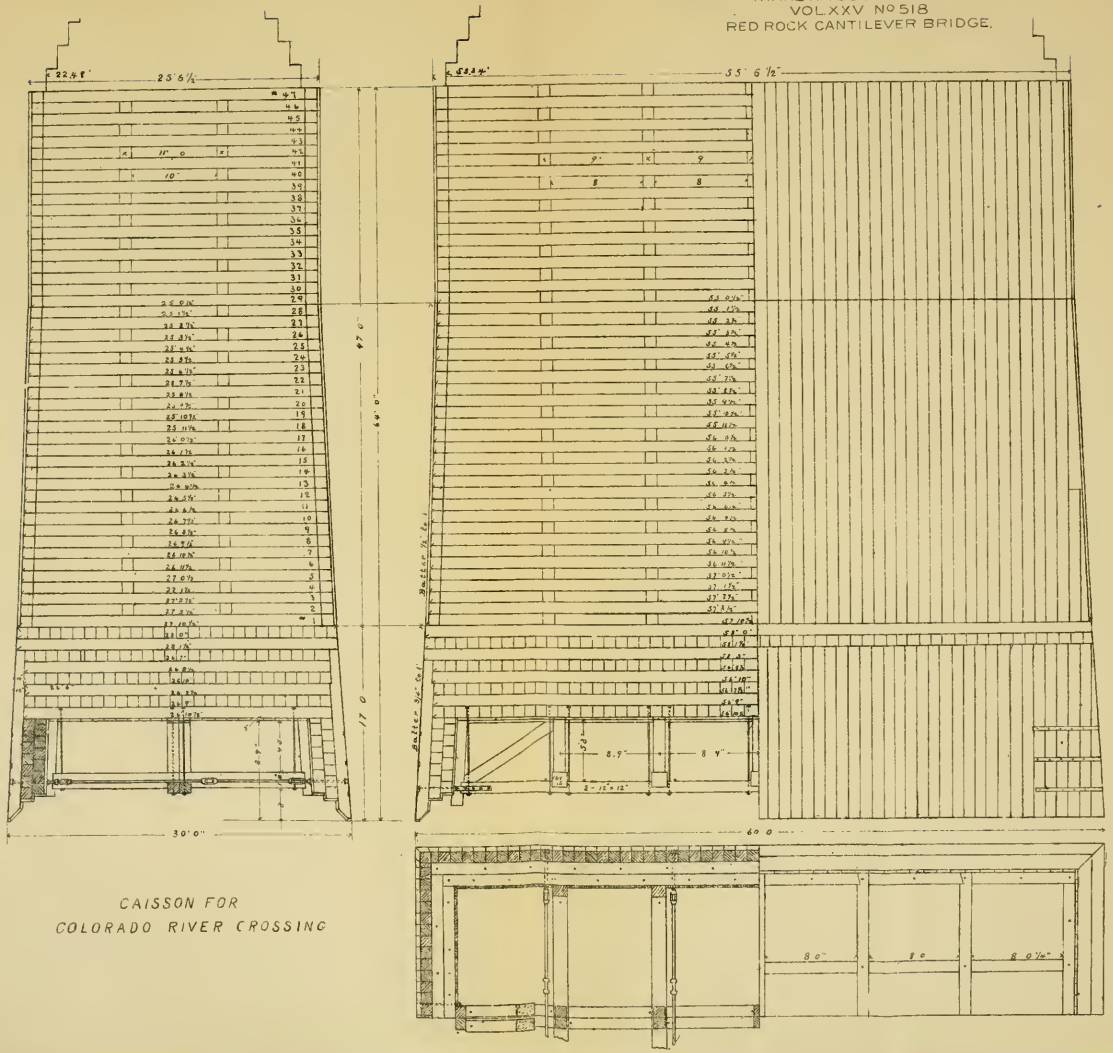
VOL. XXV Nº 518
RED ROCK CANTILEVER BRIDGE.



CAISSON FOR COLORADO RIVER CROSSING

PLATE CXIV.
TRANS. AM. SOC. CIV. ENGRS
VOL. XXV No 518
RED ROCK CANTILEVER BRIDGE.

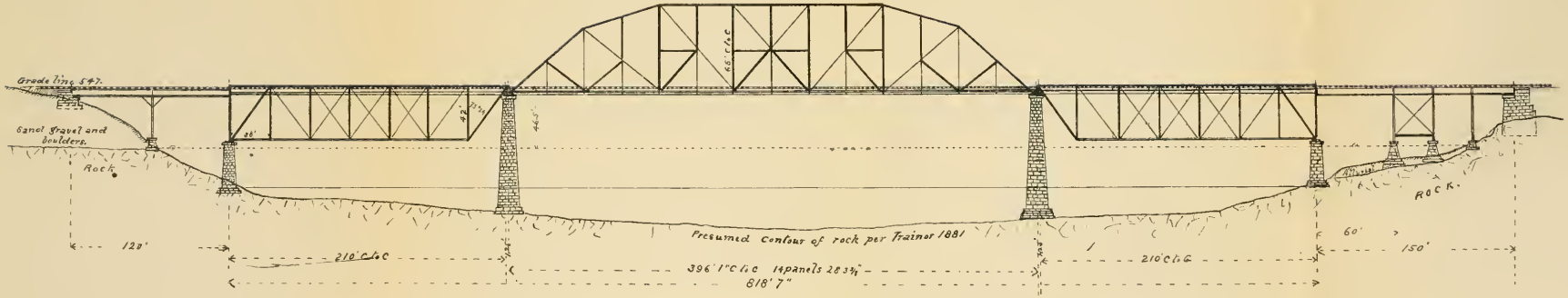


CAISSON FOR
COLORADO RIVER CROSSING

PLATE CXV
TRANS AM SOC CIV ENGRS
VOL XXV NO 518
RED ROCK CANTILEVER BRIDGE

RED-ROCK CROSSING OF THE COLORADO RIVER.

P B Co's Design, Modified.



Albuquerque Dec 15th 1888

Chief Engr.

PLATE CXVI
TRANS AM SOC CIV ENGRS
VOL XXV No 518
RED ROCK CANTILEVER BRIDGE

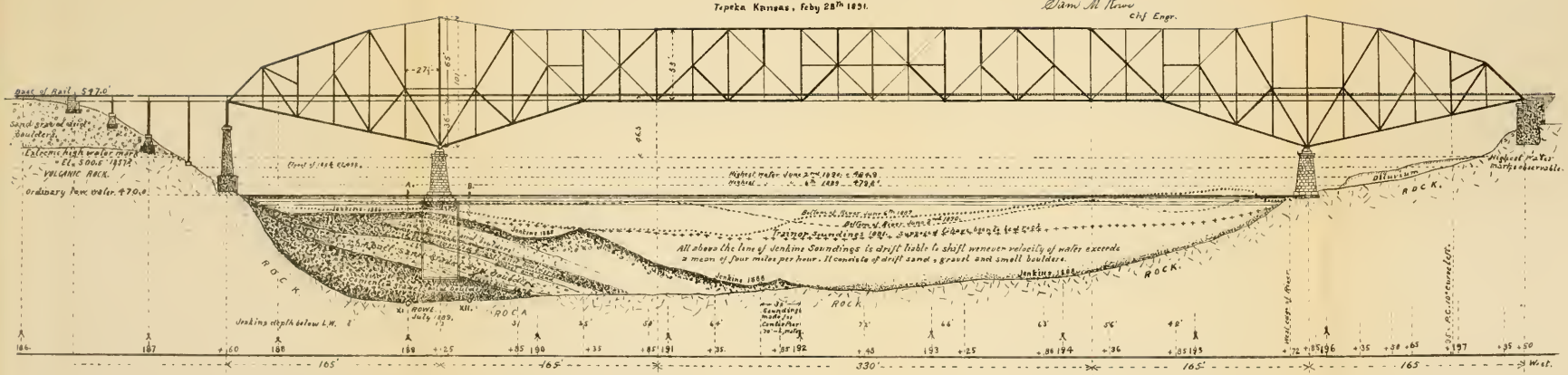
Atlantic & Pacific Railroad.

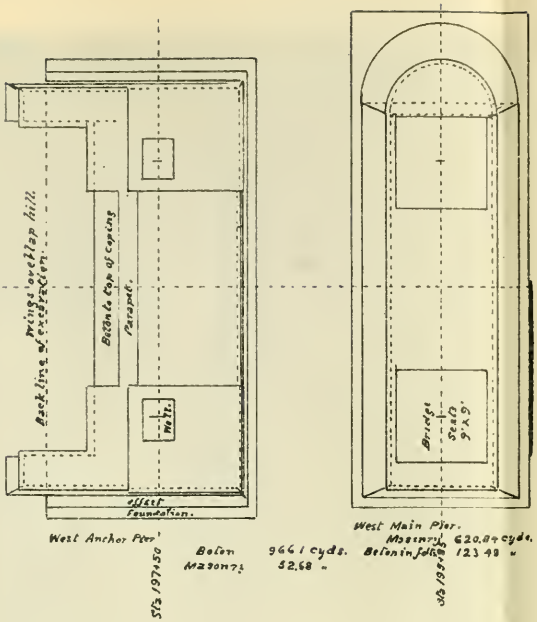
RED-ROCK BRIDGE.

(Cantilever.)

Topeka Kansas, Feby 28th 1891.

Sam'l M. Rowe
Chf Engr.



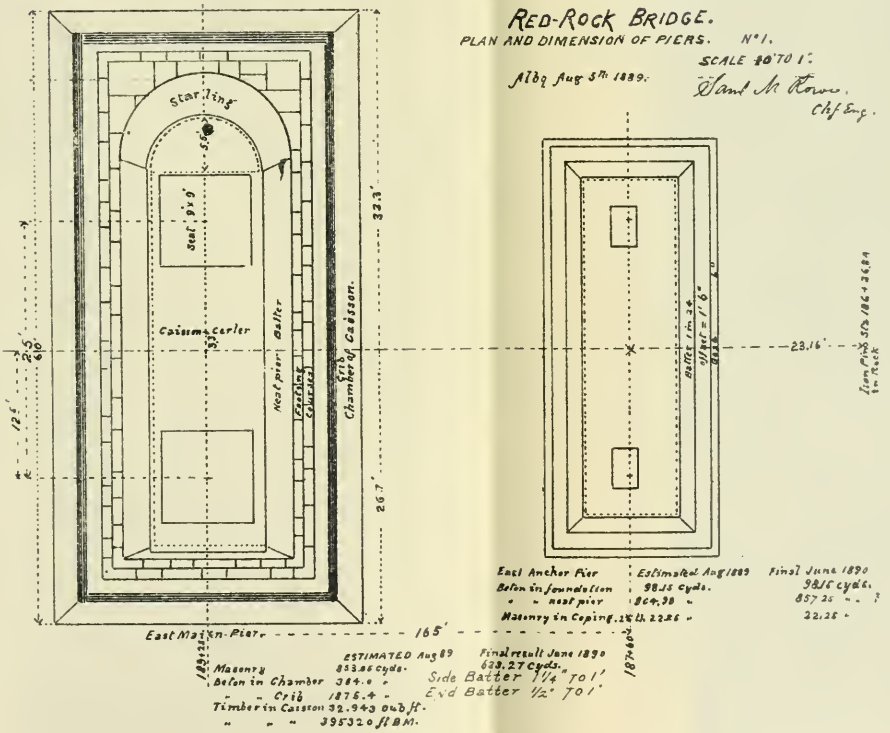


RED-ROCK BRIDGE.

PLAN AND DIMENSION OF PIERS. Nº 2

Albq Aug 5th 1889.

Scale 5' 10'



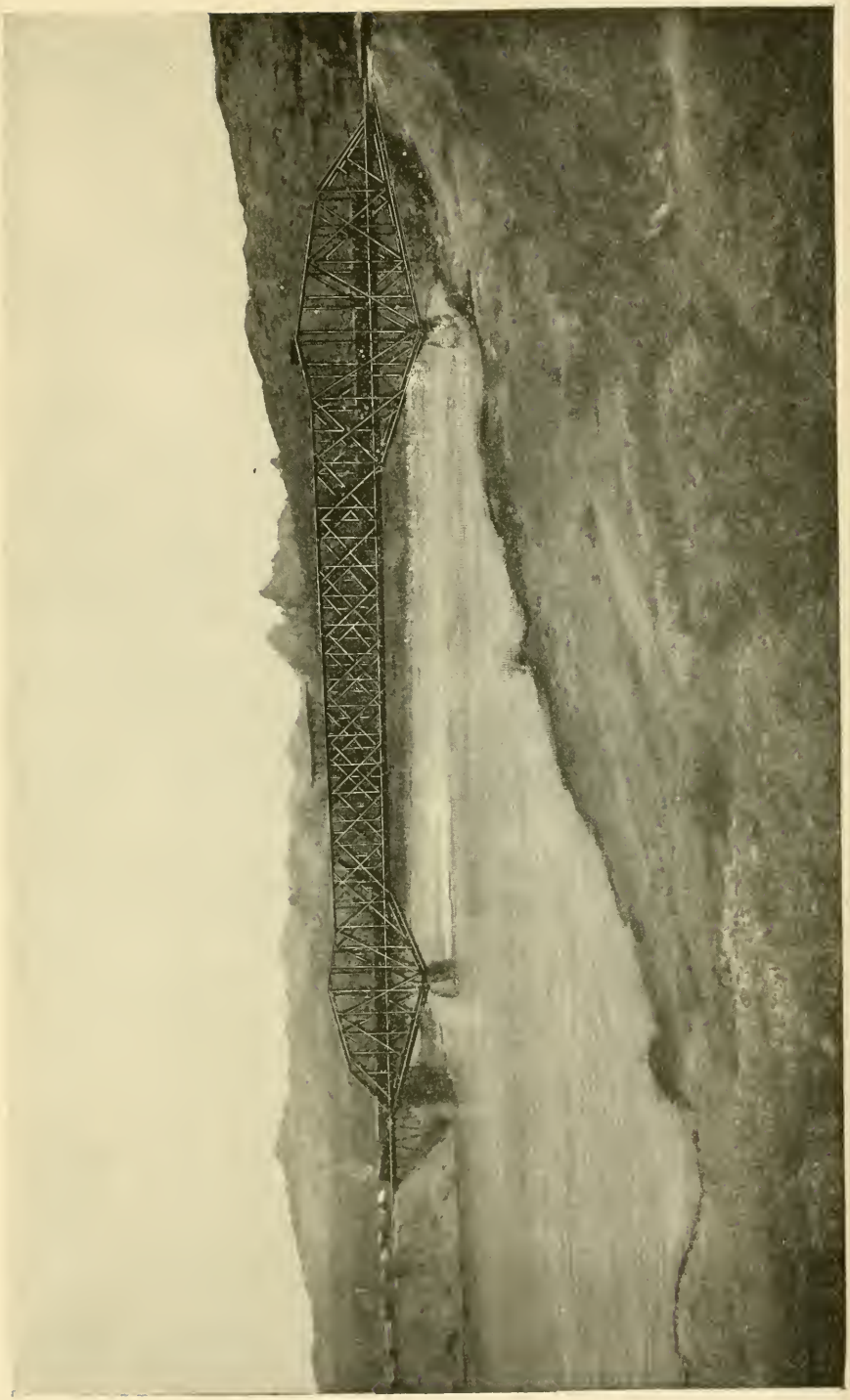
RED-ROCK BRIDGE.

PLAN AND DIMENSION OF PIERS. Nº 1.

SCALE 10 TO 1".

Albq Aug 5th 1889.

Wm M Rowe.
Chf Eng.



RED ROCK BRIDGE.

GENERAL SPECIFICATIONS AND PROPORTIONS.

By S. W. Robinson, M. Am. Soc. C. E.

Among the questions of importance concerning the Red Rock Cantilever Bridge, consideration of which was shared by me, may be mentioned the general specifications for the superstructure ; the form of structure; its economic general proportions; and the results of test of materials, and members.

Specifications.—Those adopted were those of the Phœnix Bridge Company for 1888, amended so as to provide for a heavier engine load, viz.: two of the heaviest engines of the Atlantic and Pacific Railroad, each with a forward truck having two pairs of wheels, and weighing 22 000 pounds; three pairs of drivers, weighing 92 000 pounds on 11 feet 8 inches ; and a tender weighing 74 000 pounds ; or a total of 188 000 pounds on 49½ feet. Also to provide for a full setting of the train brakes on the viaduct approach; to reduce the maximum unit strains allowed by about 6 per cent.; and for making the pins of steel of 70 000 pounds ultimate tensile strength, with duly proportioned connections. The amended specifications increased the weight of the floor system about 20 per cent. and trusses about 7 per cent. over what is to be regarded as good present practice; thus assuring ample capacity of the bridge for unusually heavy engines and loads, and the probable increased weight of rolling stock and loads of years to come.

Type of Structure.—According to the most reliable information at hand as to the river bed, when the general form of superstructure was adopted, the cantilever was agreed upon as the most suitable for this location. As expedition was too urgent to allow time for multiplied trial strain sheets seeking the most economical proportions, and to accord with relatively short anchor arms, an unusually long center suspended truss was accepted. When the most economic proportions were

settled, it was too late to make changes, had it been desirable, but the half span length given the suspended truss, was found, everything considered, about the best.

Economic Length of Center Suspended Truss.—Reason for an excessive length of this center truss was found, in the fact that the structure was to be chiefly of steel. Quite extended calculations on both a center suspended truss of half the main span length and also a third of the span length, taking account of all loads, dead, live and wind, furnished grounds for the following statements, viz.:

1st. For maximum unit strains for steel, by the formula $s\left(1+\frac{\text{min.}}{\text{max.}}\right)$ of the specifications adopted, the half span length of center was more economical than the third span length. 2d. For maximum unit strains for steel by formula (s) there was no difference in economy. 3d. For maximum unit strains for iron by formula $s\left(1+\frac{\text{min.}}{\text{max.}}\right)$ there was no difference in economy for the two lengths. 4th. For maximum unit strains for iron by formula (s) economy favors the shorter center. 5th. For iron, the most economic length of the center suspended truss is shorter than for steel. 6th. The lower the maximum unit strain allowed in a given case, the shorter will be the most economical length of center truss as compared with the main span.

In a subsequent examination of the question it was found that by the formula:

$$\text{Allowed unit stress} = s\left(1+\frac{\text{min. total stress}}{\text{max. total stress}}\right)$$ in the case of a center suspended span of $\frac{5}{12}$ the length of the main span, there was found a saving of $1\frac{6}{10}$ per cent. of steel, as compared with the case of a one-half span length of center truss. For iron it is probable that $\frac{4}{10}$ would be found a more economic ratio than $\frac{5}{12}$. The main span is 660 feet in length. The disadvantage of 1.6 per cent. found above, or about $2,000, on the structure, is probably fully met in the greater lateral stability of the bridge as due to the adopted half-span length of center truss.

Lateral Stability.—This question of lateral stability became an important one when the adopted width of about $\frac{1}{27}$ the span was assailed by those of good judgment and standing, and also in view of the fact that work on the bridge was in progress. Accordingly answers were soon given proving that the $\frac{1}{27}$ in this case was as good as the $\frac{1}{18}$ of span

for simple bridges, as adopted by good designers. Since then the question has been further investigated in a general way, and from two different points of view. The results of the investigation are here given, and establish the 25-foot width adopted as ample.

First.—For a general formula the elasticity of the laterals and of the chords was considered. In this, it is readily seen that if the laterals in the cantilever and simple bridge are at the same angular positions in plan, their elongations and compressions for maximum strains will be the same in the one as the other, so these can be disregarded.

Next, as to the chords, considering the lateral strain as due to uniform load, the strains in the chords will be similar in distribution to those due to maximum vertical loads, and the unit strain nearly uniform; so that the chords elongate or shorten very nearly a constant amount throughout, for a given case. Hence the lateral deflections due to strains in the chords would be those due to the shifting of parts in plan of the bridge from the straight line, to circle arcs of common radius. This furnishes a rational basis for calculating deflections.

Next, as to the end conditions, the fairest assumption for the cantilever arms is that they are fixed to their piers, and that the suspended truss is jointed to them. Also the simple bridge compared with, should be regarded as fixed at one end to its pier or abutment, while the other end is in effect jointed to the pier because one end usually is on expansion rollers.

Then, if l be the length of span, of either a cantilever or simple bridge, nl the length of the center suspended truss of the cantilever, b the width of the cantilever center to center, and b^1 that of the simple bridge, we find for the deflection of the end of the cantilever arm—

$$\frac{s_1 l^2}{Eb}\left(\frac{1-n}{2}\right)^2,$$

and for the center truss, its middle point compared with its ends,

$$\frac{s_2 l^2}{Eb}\left(\frac{n}{2}\right)^2$$

all parts of the cantilever being supposed deflected into horizontal circle arcs of the same curvature, as above explained.

Also of the simple bridge of length l and width b^1,

$$\frac{2 s_2 l^2}{Eb^1 (2+\sqrt{2})},$$

which last is to be placed equal to the sum of the first two expressions,

that is, the total deflection of the cantilever bridge is to be the same as that of the simple bridge, in which s_1 and s_2 are unit stresses, due to lateral loading, and E the co-efficient of elasticity, taken the same in the simple truss as for the cantilever trusses.

The last expression supposes the simple bridge, fixed at one end and supported at the other, and the curve of deflection to be composed of two circle arcs in common tangency at a point of contraflexure at a distance from the fixed end $= \frac{l}{2+\sqrt{2}}$, and which supposes the supported end to be in the direct line of the tangent to the curve at the fixed end. This relation is readily proved by trigonometry.

Hence, writing the equation we get—

$$\frac{b}{b^1} = 1.45\left(1 - 2n + 2n^2\right)\frac{s_1}{s_2}.$$

If s_1 differs from s_2, the latter is probably greatest because of the greater depth of the cantilever arm at the pier, while its breadth is the same throughout the bridge.

But taking $s_1 = s_2$ we obtain—

For $n =$	$\frac{1}{2}$	$\frac{b}{b^1} =$	.725 the minimum.
"	$\frac{1}{3}$	"	.806
"	.4	"	.754
"	.81	"	1.000

If the simple bridge with a span of 660 feet be assumed 36 feet wide, or about $\frac{1}{18}$ the span, this cantilever with a 330 feet suspended truss will have equal lateral stability when 26 feet wide for $n = \frac{1}{2}$, and for $n = .4$ when it is 27 feet wide. The bridge was built 25 feet wide, center to center, and some 29 feet wide at the out to out of the heavy chords of the cantilever arms, a width which is equivalent to about 28 feet for purposes of lateral stability or stiffness.

2d. A formula may be obtained on the supposition that the bridges in deflection will follow the same law as solid beams. Then the moment of inertia of the sections will vary as the cube of the breadth of bridge.

Hence, from usual formulas for deflection, with the end conditions as before—

$$.0054\frac{wl^4}{Eb^{13}} = \frac{wl^4}{3\,Eb^3}\,\frac{n}{2}\left(\frac{1-n}{2}\right)^3 + \frac{wl^4}{8\,Eb^3}\left(\frac{n}{2}\right)^4 + \frac{5\,wl^4}{384\,Eb^3}\left(\frac{2}{n}\right)^4;$$

or,
$$\frac{b^3}{b^{13}} = 3.86\left(n\,(l-n)^3 + \frac{n^4}{2.42}\right).$$

The minimum value of this occurs at,

$$n = 0.53 \text{ when } \frac{b}{b^1} = .695 \text{ the minimum.}$$

and for $n =$ .5 " .698
" .4 " .72
" .3 " .74

The two formulas agree essentially in results, and give the width of the bridge at practically the same figure as that adopted in the construction. These formulas indicate a slightly wider cantilever than those obtained at the time of the first investigation, owing to the fact that there the simple bridge placed in comparison was regarded as supported or jointed at both ends, while evidently it should be jointed only at one end and fixed at the other as herein considered.

The width, 25 feet, compared with the span is as 1 to 26.4, which compares favorably with several notable bridges regardless of type, viz.:

The Ohio River Bridge, Cincinnati, O	1	in	26.
Britannia Tubular Bridge	1	"	38.3
Niagara Suspension Bridge	1	"	34.
Victoria Tubular Bridge	1	"	21.
Freiburg Wire Bridge	1	"	41.
Niagara Wire Bridge	1	"	33.
Union Iron Suspension Bridge	1	"	25.

In the light of these precedents as well as from the results of analysis, there can exist no question as to the judicious selection of the width of the Red Rock cantilever.

The question of widening the cantilever at the piers without changing the center was carefully considered and advised adversely for two reasons: first, on account of the complication of shop work; and, second, from the outward thrusts at the bridge seats tending to split the pier as the cross-member at the seats expand and contract by heat and cold.

Section of Suspension and Anchor Bars.—One question which appears to have been overlooked heretofore in cantilever bridges, is that of the increment of stress in the main anchor and suspension links, due to the expansion and contraction from changes of temperature of the bridge, and consequent unavoidable springing of the links, or else slipping of them around on their pins. Evidently within certain limits the links will spring, beyond which the friction being overcome, they will slip on their pins. At first, in new bridges, the co-efficient of friction will be between 0.2 and 0.3, when later, by abrasion or by corrosion, there

will be a sticking fast of link and pin. Entire safety of the structure demands that the links be so proportioned as to spring without slipping while the strains remain within the allowed limits. Granting that the pin will never stick fast but that the co-efficient of friction, when the construction oil is exhausted, is the same as for a locomotive wheel on the rail, viz.: 0.3, the stress in the neck of the link to slip it on its pin is something surprising. Thus, equating the moment of friction with the moment of stress in the neck of a link we get—

$$tbdfr = \frac{sbd^2}{6};$$

whence,

$$\frac{s}{t} = \frac{6fr}{d}$$

in which t = the unit tension load on a link, s = the unit stress in the neck of a link due to the bending moment caused by friction, b = thickness, d = breadth of neck of link, and r = the radius of pin.

If, for example, a pin be 8 inches in diameter, the link 2 x 8 inches and the co-efficient of friction $= f = 0.3$.

$$\frac{s}{t} = 0.9,$$

that is, where the breadth of a link is the same as that of the pin on which it turns, the added stress in the link due to pin friction is $\frac{9}{10}$ that due to the suspended load; from which it is plain that any suspension or tie link intended to swing loaded on a suitable pin must be largely increased in section above that of a link subject to ordinary conditions.

But the anchor and suspension links in this Red Rock bridge were so long as not to approach the point of slipping on the pin, the whole expansion of the trusses being taken care of by the springing of the links. The superadded stress due to spring was calculated and provided for in proportioning these links. A formula for this was obtained upon the supposition that the link is fixed at both ends to its pins, and with a point of contraflexure at the middle, the half link being thus conditioned like a beam fixed at one end and free at the other. Then the well-known formulas for flexural stress and elastic deflection give

$$Pl = \frac{sbd^2}{6} = \frac{3\ EI \triangle}{l^2} = \frac{Ebd^3 \triangle}{4\ l^2};$$

whence the flexural stress,

$$s = \frac{3 \triangle d}{2\ l^2}.$$

Taking E at 25 000 000, d at 8 inches, l at 25 feet, and $\triangle$ the deflection at 0.6 inches as for the suspension links in this bridge:

$$s = 2\,000 \text{ pounds per square inch,}$$

a value less than that required to turn the link on the pin, so that the link will not turn; but the 2 000 pounds is to be included in the maximum allowed stress, and hence the links must be some 12 to 16 per cent. thicker than if this unavoidable flexural stress is neglected. The anchor links were likewise provided for, besides requiring the lengthening of those at one anchorage beyond the preferred length, to bring the flexural strains within suitable limits.

Quality of Materials and Members.—The requirements of the specifications respecting qualities were physical and not chemical, and the reports of the tests gave evidence of remarkable uniformity of test values.

The only matter worthy of note here was, 1st, the fact that two out of the eight full sized eye-bars selected for test gave much lower ultimate resistances than was expected, judging from the "heat tests." The requirements of the specifications, and results of the two full sized eye-bars in question, are grouped in the following table :

Name of Specimen.	Length, feet, of specimen.	Section of specimen in inches.	Ultimate unit tension.	Least ultimate unit tension.	Least elastic resistance limit, = ½ ultimate.	Per cent. of elongation in 10 least diameters.	Per cent. reduction of section.
Required by Specifications	....	....	62 500	58 000	½ ultimate.	20.5	41.
First eye-bar	28.5	2 x 8 inches.	....	48 345	27 570	31.6	53.1
Heat test of same	....	"	....	61 060	36 460	27.	61.
Second eye-bar	28.5	2 1/16 x 8 "	....	50 120	28 915	33.7	48.
Heat test of the same	....	"	....	63 550	37 590	26.5	62.4

For these finished eye-bars, all the test figures show higher values than those required by the specifications except for the ultimate resistance, and the fractured section was noted as silky. The heat tests show no exceptions to the specifications. Comparing the latter with the former, there is seen a general drop in values, indicating a depreciation in quality. But this very largely is to be accounted for in the much greater section under test in the eye-bar than in the three-quarter-

inch heat test specimen, lower values for large sections being well known to be usual.

Though the low ultimate tension in the finished bars was to be regretted, yet the ductility and fibrous structure were so thoroughly established as to assure confidence in the final acceptance of the bars. In another eye-bar a flaw in the head was discovered by the inspector, when the bar was set aside for test purposes. This bar broke in the flaw at about 77 per cent. of the value obtained in the neck after reheading and annealing. The fracture in the head had a crystalline appearance devoid of fiber, as if the steel were slightly burned while forging the head, the same probably being one cause of the flaw. The neck of the bar when it was broken gave an ultimate unit resistance of 56 360 pounds, which, as in case of the other two, is also somewhat lower than the specifications call for.

With some samples of pin steel there was a similar experience, partly explained, however, in the fact that some specimens were taken from a diametrical position in the bars.

RED ROCK BRIDGE.

SUPERSTRUCTURE.

By Henry H. Quimby, M. Am. Soc. C. E.

The location of the piers in determining the length of the river span and the anchor arms, gave proportions unusual, and for economy, undesirable in such structures. But the relative shortness of the anchor arm is of manifest advantage in securing uniformity in the character of the strains in all the members of that portion of the truss. The dead weight of the structure between the piers is sufficient to overbalance the combined live and dead load on the anchor arm, and by consequence, reverse strains cannot occur in any member. This condition would permit the use of tension members in the upper chord of the anchor as well as cantilever arm and the end braces, but it was deemed advisable to make each anchor arm a complete truss competent to support itself and

the heavy traveler in the event of disaster to the falsework. Therefore the upper chord panels affected, and the end braces, were built of 24-inch latticed chords and 8-inch eye-bars in combination. The trusses were at first designed to have the top chord level over the piers and river span, but the change, increasing the depth 10 feet over the piers, relieved the long inclined posts of a considerable portion of their load and effected a better distribution of the pressure on the pin and shoe, besides giving a more pleasing outline to the bridge (Plate CXXIII).

Facility of erection had to be kept constantly in mind in designing the arrangement and connection of members, and in some cases the strains produced by the traveler determined the section. The members in the top and bottom panels between the center span and lever arm, have no other duty than that of forming the chords of the lateral trusses. In the erection, however, they were subject to tensile and compressive strains about equal to those in the corresponding panels of the great Forth Bridge. The computations were based on the following loads:

Live.—Two engines each weighing, with tender, 94 tons, concentrating 46 tons on a wheel base of 11 feet 9 inches, and followed by 3 000 pounds per foot.

Dead.—Track, 450 pounds per linear foot; floor system, 450 pounds per linear foot; trusses of anchor and lever arms, varying from 3 200 to 4 100 pounds per foot; trusses of center span, 1 550 pounds per foot.

Wind.—Thirty pounds per square foot on exposed surface of a train and both trusses.

The allowed unit stresses as prescribed by the Consulting Engineer, Prof. S. W. Robinson, were in general somewhat lower than those of the Phœnix Bridge Company's standard specifications, under which in all other respects the bridge was designed, and were for—

Steel eye-bars and verticals 9 200.

Diagonals and chords $9\,400\left(1+\frac{\text{min.}}{\text{max.}}\right)$ whenever this quantity exceeds 12 300.

Plate hangers 8 000 pounds.

Compressiom members, flat ends, $P=\dfrac{10\,000}{1+\dfrac{l^2}{40\,000\,r^2}}$.

Compression members, pin ends, $P=\dfrac{9\,730}{1+\dfrac{l^2}{30\,000\,r^2}}$.

Pins and rivets, shearing.....	9 000 pounds	for combined live and dead loads.
	12 000 "	for wind and erection.
	11 000 "	for wind and live in combination.
bearing	14 500 "	for combined live and dead.
	20 000 "	for wind or erection.
	18 000 "	for wind and live load in combination.
bending on extreme fibers....	20 000 "	for combined live and dead loads.
	26 000 "	for wind or erection.
	24 000 "	for combined wind and live.

For field connections the number of rivets to be increased 25 per cent.

The material used throughout, except in the floor, adjustable rods, upper struts, and lacing of chords, is open hearth steel, for which the requirements were:

Ultimate—for tension steel—between 58 500 and 66 500.
for compression—between 64 000 and 72 000.

Elastic limit—not less than one-half the ultimate tensile strength.

Elongation in 8 inches—not less than $\frac{1\ 200\ 000}{\text{ult.}}$ per cent.

Reduction of area—not less than $\frac{2\ 400\ 000}{\text{ult.}}$ " "

The erection stresses were due to the weight of the overhanging structure with the track and the traveler, the latter weighing 240 000 pounds, which, with the weight of pieces being raised in position, was all concentrated on the front wheel. The resulting strains in the top panel of the lever arm (Plate CXXIV) were twice as great as those to be provided for in the finished structure, and these panels were consequently reinforced during erection by the addition of a double line of $2\frac{1}{4}$-inch square iron bars on the midline of the truss. Temporary diagonals, made of adjustable rods and stirrups, were employed to support the middle of the end post of the center span while the traveler was at that point. All the joints of the top chords of the center span were proportioned for the tension during erection and were made forward of the pins. The bottom chord panels, subject to compression in erection, were made of 24-inch built channels, latticed.

Each truss is anchored (Plate CXXV) with four $8 \times 1\frac{9}{16}$-inch eye-bars secured by an 8-inch pin to a 24-inch bolster, 8 feet long, which carries one end of two 50-inch plate girders extending the whole length of the piers.

On these girders rest fifteen 15-inch rolled beams embedded in the mass of concrete. At the west end the bars are connected to the truss heel by a short, stiff member which also carries the end floor beam. At the east end sufficient anchor weight was secured in the pier without carrying it up to the truss, and for economy the concreting was stopped 14 feet 6 inches below the heel pin, the anchor bars in the intervening space being made of 24-inch built channels braced transversely by heavy latticed diagonal struts, designed to transmit to the top of the pier the horizontal wind thrust from the trusses. The connection to the pier top is by sixteen bolts $1\frac{7}{8}$ inches diameter in slotted holes, permitting no lateral motion, but a slight longitudinal as well as vertical movement due to thermal changes and deflection. At the west end the floor beam, while not resting on the pier, is bolted to it with similar liberty of movement.

The pedestals on the main piers are each 7 feet 3 inches square, and consist of eight ribs in a transverse direction united by heavy top and bottom plates. The truss shoes resting on the pedestals have four longitudinal ribs with a total thickness of 12 inches, which forms the bearing of the 15-inch main pin on which abut the five principal compression members.

The center span is suspended by four 8 x $2\frac{5}{16}$-inch eye-bars from the upper end of the lever arm, and swings freely in oblong pin-holes at both ends. The floor beams throughout are riveted to the vertical members. In the anchor and cantilever arms the stringers, being near the neutral axis of the truss, are riveted to the webs of the floor beams; but in the center span, where they are in the plane of the lower chord, they are carried by brackets on the floor beam, and provided with slotted holes to permit the slight longitudinal motion caused by the stretch of the lower chord consequent upon the deflection of the trusses.

The transverse and horizontal bracing (Plate CXXVI) is of latticed struts and upset rods, with clevis attachment, except between the main inclined posts and along the lower chord of the anchor and cantilever arms, where the diagonals also are lattice struts, in the latter case 30 inches deep. The main inclined posts and all the verticals in the arms have transverse bracing below the floor. The verticals over the piers are 101 feet long, with one splice, and the inclined ones adjacent, 107 feet, also made in two lengths. Those in the cantilever arm, having an accumulation of

top wind load to transmit to the pier, are stiffened against bending at the point of attachment of the transverse bracing. The pins generally are 8 inches in diameter, and the shoe pins (Plate CXXVII) are 15 inches in diameter, 5 feet long, and weigh 3 000 pounds each.

The lower lateral system of the center span ends, and that of the cantilever arm, begins at the point where expansion takes place, the shear being transmitted from one system to the other by lateral contact, one member sliding within the other. Although all the wind load from the top chord of the center span is assumed to pass down the end braces into the lower system, as (being its most direct road to the pier) the upper system of rods is carried through the expansion panels and a similar device employed to transmit shear.

The track rails are steel, 61 pounds per yard, with ordinary four-hole angle splices and spiked with $\frac{9}{16}$-inch square x $5\frac{1}{2}$-inch spikes. The cross-ties are white oak 8 x 12 inches x 10 feet, set on edge, $13\frac{3}{4}$ inches center to center, and notched three-quarters of an inch over stringers. On each floor-beam is a tie laid flat. Every third tie is 16 feet long, carrying two walks, which are each made of two 3 x 12-inch planks 9 inches apart. The outer guards are 6 x 6 inches Southern pine, notched $1\frac{1}{2}$ inches over ties, spliced with an 8-inch scarf, and bolted through every third tie to each stringer by a $\frac{3}{4}$-inch bolt passing through the stringer flange. These guards are directly over the stringers, 8 feet center to center. The inner guards, which have not yet been put on, are designed to be 4 x 4 x $\frac{1}{2}$-inch angle iron set 8 inches from the rail head and bolted with $\frac{5}{8}$-inch lag screws to every tie.

The calculation of the lengths of members was made with a view of having them in their normal position when fully loaded. In the center span the trusses were cambered 3 inches by increasing the top chord panels and the diagonals proportionately, adding to compression members and deducting from tension members the compression and stretch due to the dead load only, using a modulus of elasticity of 26 000 000 pounds. For the arms the lengths were figured with all the pins at their normal elevation, but the allowance for compression and stretch covered the whole load—live and dead. The effect on the arms was to shorten the whole top chord and lengthen the whole bottom chord, consequently raising the end of the cantilever arm. The allowance in the long inclined posts increased their length $\frac{7}{16}$ of an inch, and the hangers depending from their tops were shortened one-quarter of an inch. The

members at this point form a rigid quadrilateral, tied diagonally, and with the allowances acting in opposition, the connecting of those in the lever arms necessitated the opening of the first joint in the lower chord sufficiently to raise all the members meeting at the lower end of the shortened hanger, until the connecting pin could be driven. The joint closed gradually as the traveler moved forward, and when the end of the cantilever arm was reached, enough strain had been produced to compress and stretch the members sufficiently to close the joint completely.

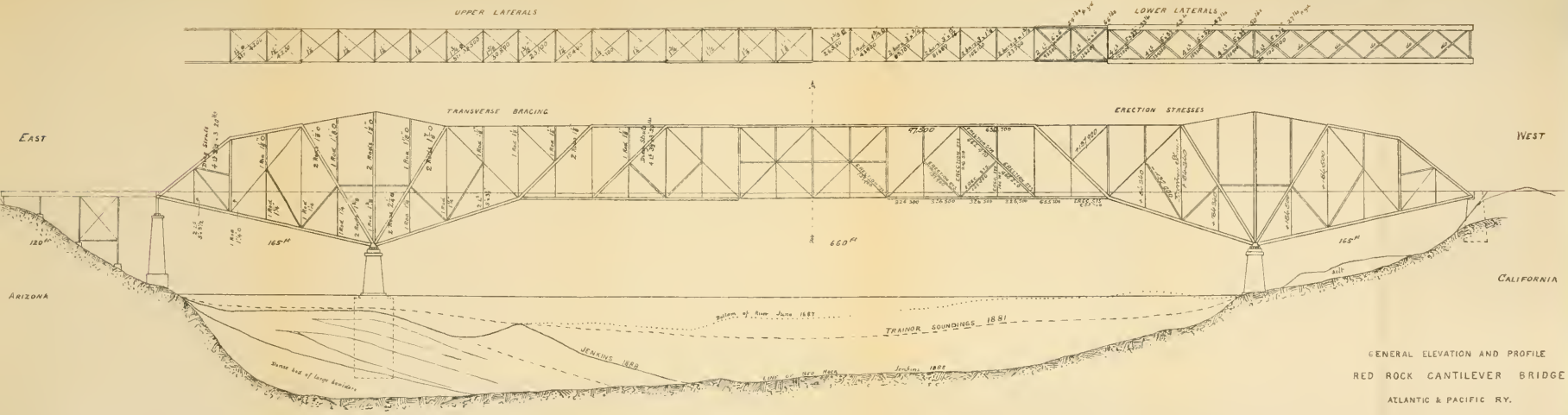
Some uncertainty was felt as to the accuracy of the setting of the pedestals on the piers as affecting the length of span. They were the starting points of the erection on their respective sides, and it would be impossible to shift them after the projecting arms were built. The great distance that the bridge is from the works, requiring four weeks for the transport of freight, made it impracticable to leave a panel at the center of the span to be made after the two halves had almost closed the gap and the length of the closing panel measured. So the shop work was completed and provision made in the adjustment panel for a possible error in length of span. The stringers in these panels (which are the ones immediately adjoining the center span), are carried in deep pockets on the floor beams, and have oblong holes at each of their ends. The pin-holes at one end of the chords in these panels were made with 9-inch play. The adjustment was by wedges (Plate CXXVII) sliding between planed surfaces, the ordinary device of working the wedges between rollers being impracticable because of the great pressure. The wedges were built of seven-eighths-inch vertical plate ribs and seven-eighths-inch plate faces united by angle iron, with their surfaces polished. The slope of each face was 1 in 12, making the wedge 1 in 6. The angle of rest with lubricated surfaces being about 1 in 8, as shown by recorded experiments, the wedges should squeeze out under pressure, and to prevent this and to move them in or out, as occasion might require, 3-inch screws were passed through and secured to them, with nuts bearing on an iron frame underneath and wooden blocking above. The lower wedges bore against the projecting beveled end of the fixed chord section, and a loose beveled box pinned to the movable section, the latter being pivoted, adjusted itself to the slope of the wedge. The upper wedges each worked between two pivoted boxes, so arranged that the operation of driving in the wedge would shorten the panel and th

whole projecting top chord, at the same time raising the center. The faces of the boxes were seven-eighths-inch plates, supported by horizontal ribs. In erection, the wedges were all set in far enough to avoid the necessity of driving afterward, although screws were provided to make this possible if the occasion should arise. The lower wedges were entered to increase the panel length 3 inches, thereby raising the center point 9 inches above the normal level, at the same time permitting a connection at the center if the span should prove to be as much as 6 inches too long. The upper wedges, similarly set, gave additional elevation at the center, and allowed for a possible shortage in the length of span. The dead pressure on each wedge was 656 000 pounds, or 910 pounds per square inch of bearing on the lower wedges, which are 30 inches wide, and 1 370 pounds per square inch on the upper wedges, 20 inches wide. As all the wind load on the projecting half span was assumed to pass through the lower wedges (that from the top chord passing down the end braces), the pressure per square inch on the latter might be increased to 1 200 pounds.

As the surfaces were to remain in close contact for several weeks in a hot climate, with the sun's heat tending to melt out the grease, and with no possibility of introducing any after setting, a mixture of plumbago, with just enough tallow to hold it in mass, was spread thickly over the sliding surfaces and proved effective in preventing abrasion.

The anchor arms were erected on falsework (Plate CXX). The pedestals and shoes were first set and then the lower chords and struts were placed in position by a small traveler. All the remaining material was handled by the large traveler, which carried three hoisting engines and erected the trusses and floor from its overhanging front, and had two booms at the back to erect the bracing behind it. Eight of the track stringers were reinforced with cover plates and fitted with rails to form a track for the traveler. These rested on top of the floor beams, which were carried by the stiffened hangers, those of the anchor arm acting as posts. The permanent track was laid as the traveler advanced, and was used for running out the material which was passed between the traveler legs to the front.

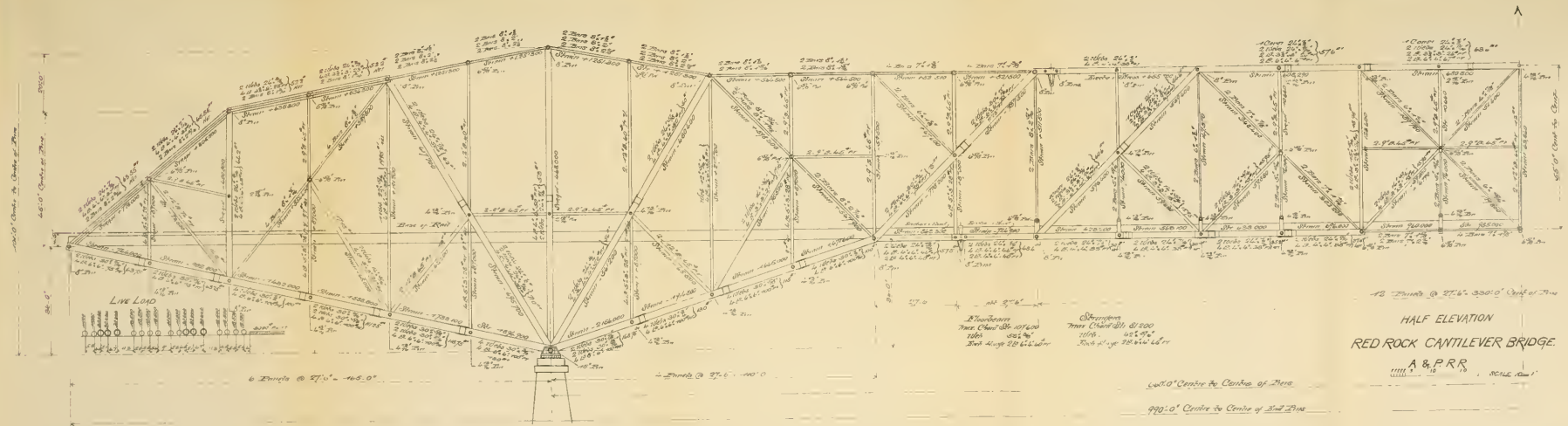
The most interesting part of the erection was the final connection at the center. For six weeks the west half of the structure had been standing with 330 feet projecting over the river, and when the east half reached it, the pin holes in the lower chord eye-bars lapped 5½ inches; showing, as the wedges had pushed each half 3 inches out, that the span



GENERAL ELEVATION AND PROFILE

RED ROCK CANTILEVER BRIDGE

ATLANTIC & PACIFIC RY.



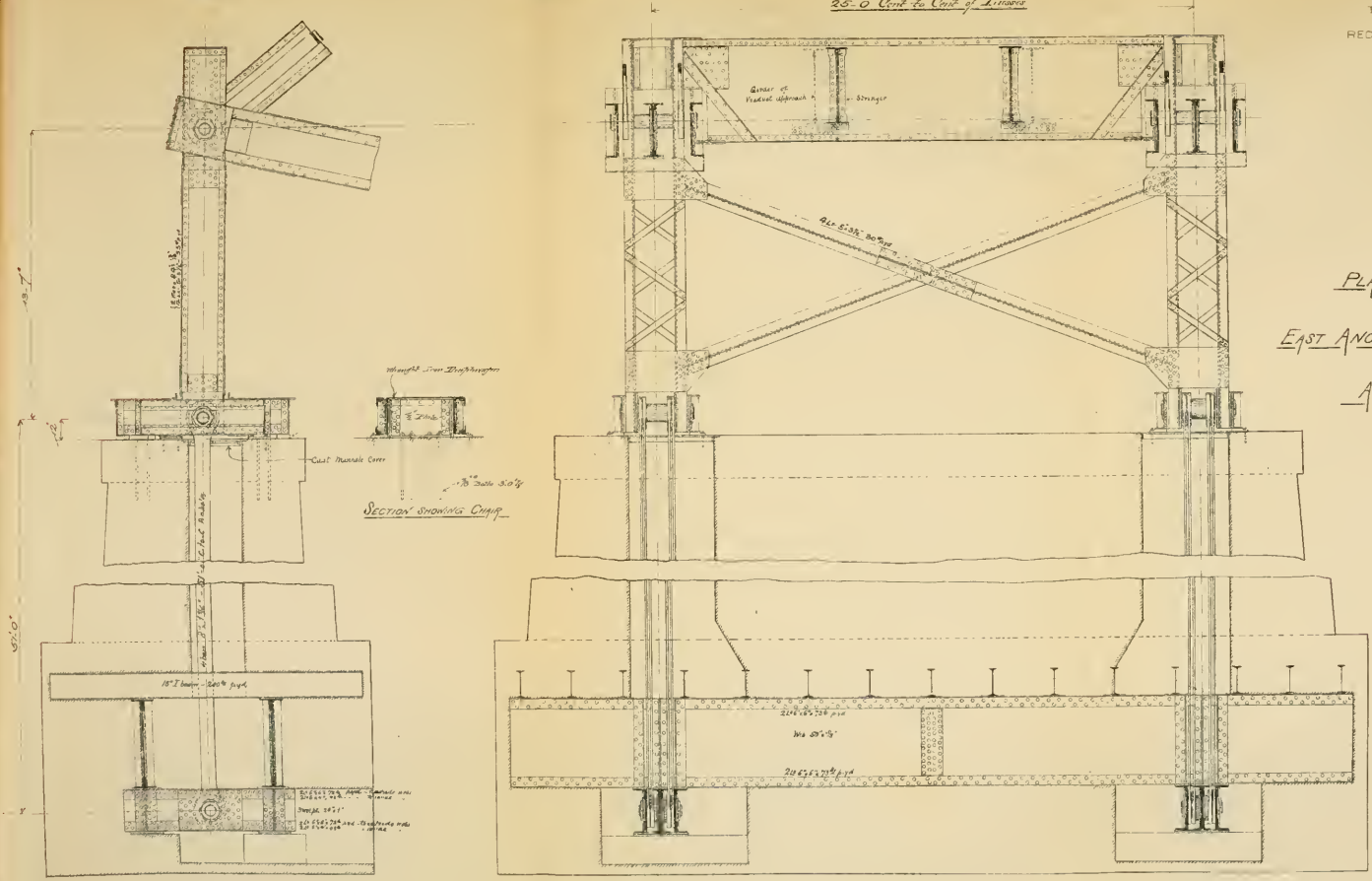
HALF ELEVATION

RED ROCK CANTILEVER BRIDGE.

A. & P. R. R.

SCALE

PLATE CXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXV. No. 518.
RED ROCK CANTILEVER BRIDGE.

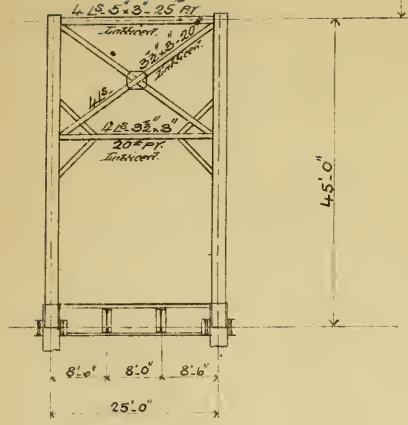


PLAN OF ANCHORAGE

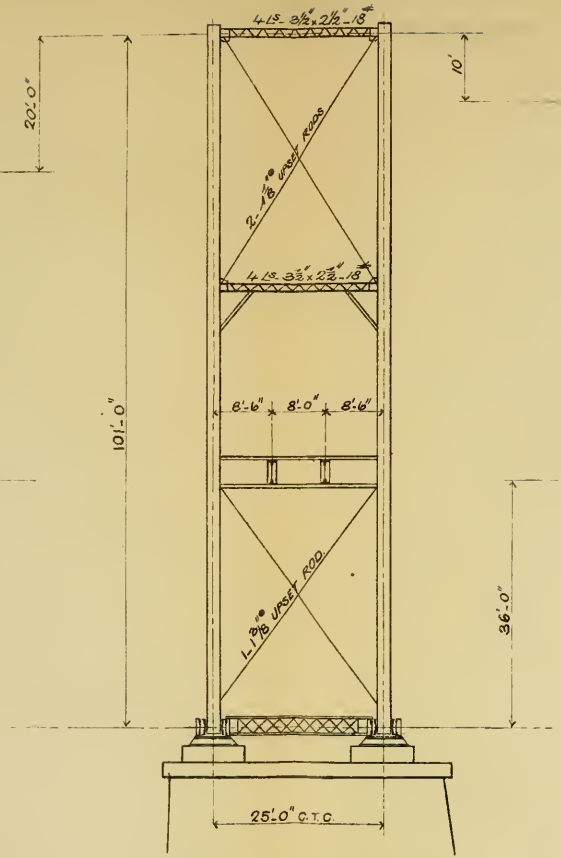
EAST ANCHOR PIER. RED ROCK BRIDGE

A. & P. R. R.

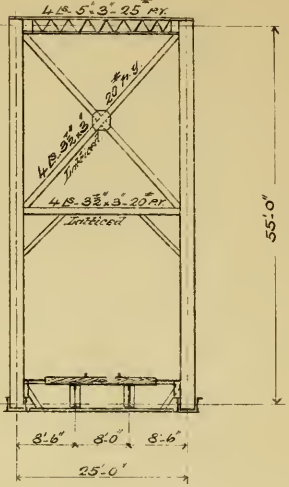
PLATE CXXVI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXV. No. 518
RED ROCK CANTILEVER BRIDGE.



END PORTAL.



TRANSVERSE SECTION
OVER MAIN PIER.

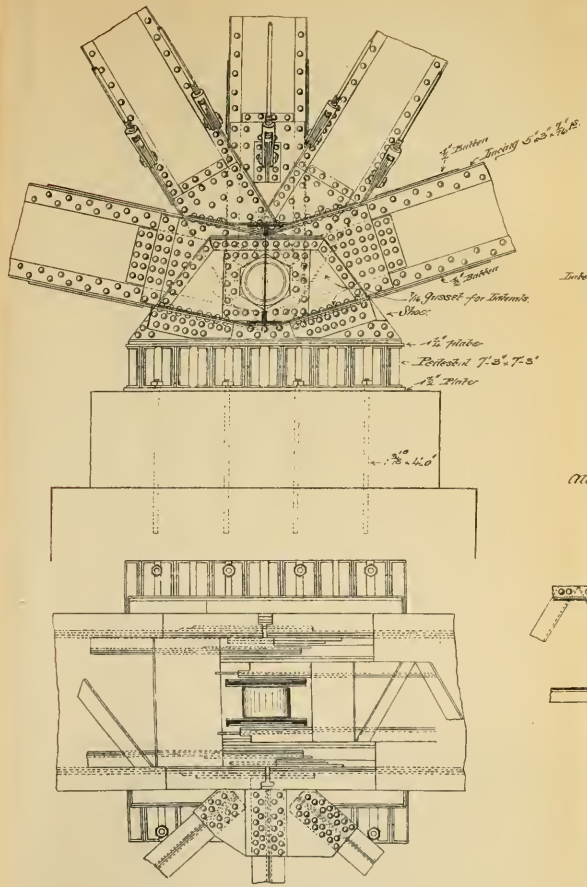


PORTAL OF SUSPENDED SPAN.

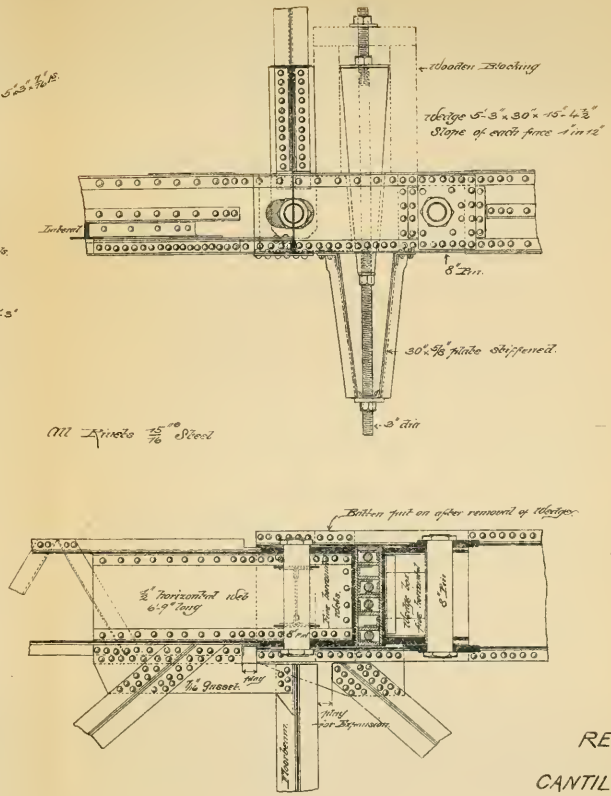
RED ROCK
CANTILEVER BRIDGE.
A. & P. R.R.

5' 10' 10'

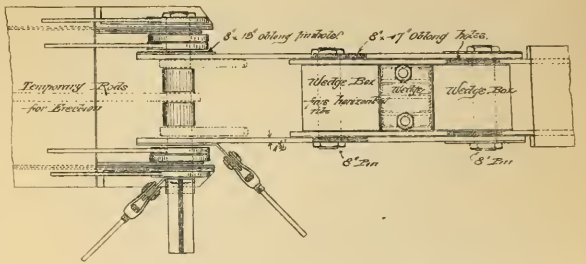
PLATE CXXVI
TRANS AM SOC CIV ENGRS
VOL XXV No 518
RED ROCK CANTILEVER BRIDGE

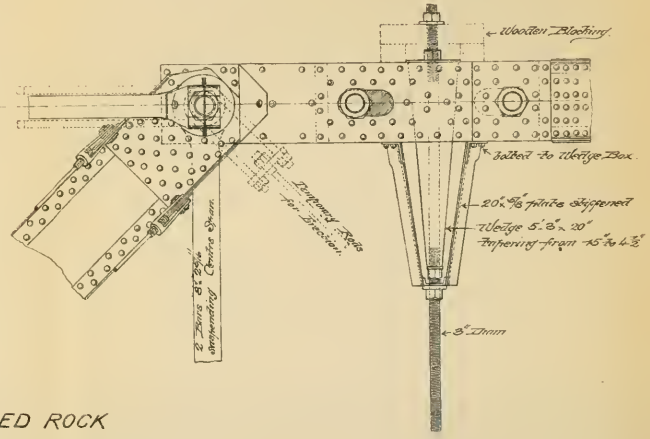


DETAIL OF SHOE & PEDESTAL.



DETAIL OF LOWER WEDGE.





DETAIL OF UPPER WEDGE.

RED ROCK
CANTILEVER BRIDGE
A. & P.R.R.

3" 3" 6" 12" 12" 12"

length, as fixed in setting the pedestals, was correct within a fraction of an inch. The process of withdrawing the wedges was at once begun and continued until the connecting pins were all driven and the wedges entirely loose, the chords of the center span taking their normal stresses. Some of the wedges moved up and out as fast as the lower nuts were unscrewed. Others had to be started by a light blow after the nuts had been moved a little.

The day after the connection was made and before the traveler was taken down, a washout at the old crossing of the river stopped all traffic on the road, and on the day following the washout, the track over the center was laid and cleared, and trains passed between the legs of the traveler, which had been lightened by the removal of engines, rigging and lower bracing. The structure thus carried a concentrated load of 110 tons at the center of the span in addition to the regular live load. Their mission ended, the wedges, bearing boxes and pins were removed, and the separate portions of each upper adjustment panel riveted together to form one member, its length being fixed according to the temperature at the time.

APPENDIX "A."

Record of Red Rock Caisson—Sinking.

Date.	Elevation of River.	Height of Caisson.	Elevation of Top of Crib at Corners.				Mean Ele. of Cutting E.	Position.				Progress.
			Northeast.	Northwest.	Southeast.	Southwest.		North.	South.	East.	West.	Feet.
1889.												
Nov. 16th.....	469.80	13.00	481.60	481.58	481.58	481.56	468.56	$\frac{5}{16}$" W.	$\frac{1}{2}$" W.	$\frac{3}{4}$" N.	$1\frac{1}{8}$" N.	1.24 Blkg.
" 17th.....	469.70	15.77	484 29	484.29	484.36	484.30	468.54	"	"	"	"	0.02
" 19th.....	469.70	16.75	485.20	485.23	485.27	485.27	468.49	"	"	"	"	0.05
" 20th.....	469.60	18.73	487.22	487.21	487.24	487.24	468.49	"	"	"	"	0.00
" 21st	469.55	19.70	488.17	488.19	488.23	488.20	468.49	"	$\frac{3}{4}$" W.	$\frac{1}{2}$" W.	No Obs.	0.00 Air, int.
" 22d.....	469.45	19.70	487.39	487.56	487.27	487.34	467.69	"	"	"	"	0.80
" 23d.....	469.57	19.70	486.13	485.93	485.88	485.97	466.28	$1\frac{3}{4}$" W.	1" E.	"	"	1.41
" 24th.....	469.70	19.70	484.59	484.47	484.42	484.50	464.80	$1\frac{1}{4}$" "	"	"	"	1.48
" 25th.....	469.77	21.70	484.45	484.06	484.30	484.06	462.52	0.0	$\frac{1}{2}$" W.	$\frac{1}{4}$" N.	"	2.28
" 26th.....	469.81	22.70	484.98	484.88	484.68	484.77	462.13	0.0	$\frac{1}{4}$" W.	$1\frac{1}{8}$" N.	"	0.39
" 27th.....	469.90	23.70	484.72	484.72	484.49	484.58	460.83	0.0	"	"	"	1.30
" 28th.....	469.88	24.70	484.76	484 94	484.58	484.94	460.10	0.0	"	$1\frac{1}{4}$" N.	"	0.73
" 29th.....	469.85	27.70	487.14	487.13	487.23	487.38	459.53	0.0	"	"	"	0.57
" 30th.....	469.80	27.70	486.70	487.70	487.80	487.96	459.10	0.0	"	"	"	0.43
Dec. 1st	469.75	27.70	486.33	486.22	486.18	486.25	458.55	$1\frac{3}{4}$" W.	$1\frac{1}{4}$" W.	$1\frac{1}{8}$" N.	"	0.55
" 2d.....	469.81	27.70	485.80	485.44	485.70	485.48	457.90	$3\frac{1}{4}$" W.	$3\frac{1}{2}$" W.	1" N.	"	0.65
" 3d.....	470.00	30.60	488.01	487.74	487.78	487.61	457.11	"	"	"	"	0.79
" 4th	470.50	30.60	487.09	486.90	486.59	486.55	456.18	1" W.	3" W.	$\frac{1}{8}$" N.	"	0.93
" 5th.....	470.70	30.60	486.34	486.38	485.78	485.85	455.50	"	"	"	"	0.68
" 6th.....	470.60	31.60	486.27	486.37	486.18	486.37	454.70	"	"	2" N.	"	0.80
" 7th.....	470.65	33.60	487.61	487.46	487.49	487.45	453.93	"	"	"	"	0.80
" 8th.....	470.70	33.60	487.08	487.03	486.89	486.97	453.39	"	"	"	"	0.51
" 9th.....	472.00	33.60	486.31	486.08	486.09	486.01	452.52	"	"	"	"	0.87
" 10th.....	471.00	33.60	485.72	485.41	485.26	485.41	451.85	"	"	"	"	0.67
" 11th.....	471.50	33.60	485.09	484.71	484.56	484.34	451.08	"	"	"	"	0.77
" 12th.....	471.80	35.60	486.20	486.11	485.13	485.16	450.05	"	"	"	"	1.03
" 13th.....	472.30	35.60	484.56	484.57	484.75	484.89	449.09	"	$1\frac{3}{4}$" W.	1" N.	"	0.96
" 14th.....	472.20	35.60	484.59	484.54	484.77	484.85	449.09	"	"	"	"	0.00
" 15th.....	472.00	35.60	483.78	487.94	483.71	484.04	448.27	"	"	"	"	0.82
" 16th.....	472.00	35.60	482.58	482.78	482.55	482.92	447.11	$2\frac{1}{2}$" W.	1" W.	$3\frac{3}{4}$" N.	"	1.16

" 17th.....	474.50	37.60	483.97	484.09	483.60	483.87	446.28	2½" W.	1" W.	3¾" N.	No Obs.	0.83
" 18th.....	471.50	37.60	483.33	483.42	482.95	483.21	445.62	"	"	"	"	0.66
" 19th.....	470.85	39.60	484.40	484.47	483.84	484.06	444.59	2" W.	0.0	5" N.	"	1.03
" 20th.....	471.20	39.60	483.37	483.35	483.35	483.48	443.79	"	0.0	"	"	0.80
" 21st	470.90	39.60	482.59	482.53	482.58	482.65	442.99	"	½" E.	3⅞" N.	"	0.80
" 22d.....	471.20	39.60	482.16	482.01	481.99	482.11	442.44	"	"	"	"	0.55
" 23d.....	470.80	41.60	483.25	483.26	483.09	482.96	441.54	"	"	3½" N.	"	0.90
" 24th.....	470.70	42.60	483.43	483.43	483.35	483.45	440.79	"	"	"	"	0.75
" 26th.....	471.00	44.60	484.23	484.23	484.22	484.22	439.60	"	"	"	"	1.19
" 28th.....	471.40	44.60	482.59	482.59	482.81	482.92	438.12	"	"	"	"	1.48
" 30th.....	471.00	44.60	480.67	480.67	480.93	480.96	436.17	0.0	0.0	4¾" N.	"	1.95
1890.												
Jan. 1st	470.40	48.60	482.97	482.89	482.95	482.90	434.33	0.0	0.0	"	"	1.84
" 2d.....	470.50	48.60	481.42	481.43	481.33	481.33	434.78	0.0	0.0	"	"	1.55
" 4th.....	470.90	49.60	480.79	480.83	480.75	480.80	431.19	0.0	0.0	"	"	1.59
" 6th.....	471.10	52.60	481.37	482.00	482.17	482.27	429.48	0.0	0.0	"	"	1.71
" 8th.....	470.50	53.60	480.78	480.85	481.00	481.08	427.33	½" W.	1" E.	"	"	2.15
" 10th.....	470.40	53.60	478.75	478.87	479.01	479.11	425.36	1" W.	¼" E.	No Obs.	3¾" N.	1.97
" 13th.....	470.15	58.60	481.66	481.75	481.91	481.99	423.23	"	"	"	"	2.13
" 15th.....	470.40	58.60	479.58	479.58	479.92	479.92	421.15	"	1¼" E.	"	3" N.	2.08
" 17th.....	470.45	58.60	477.62	477.55	477.89	477.79	419.11	1½" E.	1⅝" E.	"	4½" N.	2.04
" 18th.....	470.40	59.66	477.00	476.83	477.38	477.27	417.46	2⅛" E.	2⅜" E.	"	4¼" N.	1.65
" 20th.....	470.35	59.66	475.31	475.08	475.35	475.18	415.57	"	"	"	"	1.89
" 21st	470.30	59.66	473.60	473.25	473.76	473.47	413.86	"	⅝" E.	"	"	1.71
" 22d.....	470.10	59.66	472.81	472.49	472.99	472.71	413.09	"	"	"	"	0.75
" 23d.....	469.90	59.66	471.82	471.51	471.93	471.67	412.07	"	"	"	"	1.02
" 24th.....	469.70	63.81	475.15	474.93	475.12	474.95	411.23	"	"	"	"	0.84 Crib Comp.
" 25th.....	469.70	63.81	474.52	474.27	474.33	474.16	410.51	"	"	"	"	0.72
" 27th.....	469.80	63.81	474.12	473.90	473.50	473.70	409.99	"	"	"	"	0.52
" 28th.....	471.95	63.81	473.76	473.51	473.24	473.16	409.61	"	"	"	"	0.38
" 29th.....	470.70	63.81	473.44	473.20	473.04	409.41	409.41	"	"	"	"	0.20
												34.36
												25.29
												59.65

APPENDIX "B."

WEIGHT OF METAL IN BRIDGE, BY HENRY H. QUIMBY.

	IRON.	STEEL.	TOTAL.
East anchorage, exclusive of floor beams	27 400	51 035	78 435
West anchorage, exclusive of floor beams	27 100	65 388	92 488
Floor of anchor and cantilever arms, including 16 600 pounds of reinforcing stringers for use in erection	112 940	158 570	271 510
Two anchor arms	227 260	742 610	969 870
Two cantilever arms	223 270	696 070	969 870
Metal over piers (vertical posts, pins, transverse rods and struts)	65 830	112 960	178 790
Expansive panels (chords and x posts)	18 470	109 700	128 170
Temporary members (wedges, reinforcing bars, etc.)	61 570	14 470	76 040
Suspended span	399 375	302 600	701 975
Total iron, pounds	1 163 215		
Total steel, pounds		2 253 403	
Total metal, pounds			3 416 618

APPENDIX "B"—(*Concluded*).

COMPUTATION OF PRESSURE ON PIERS, BY HENRY H. QUIMBY.

Anchor arm, including iron floor	620 700
Cantilever arm, including iron floor	659 600
Anchorage reaction (excess of cantilever over anchorage)	30 000
Pedestal posts and bracing over pier	89 400
Suspended span, including anchorage reaction	702 000
Track, at 450 pounds per foot, including anchorage reaction.	297 000
Live load, at 3 000 pounds per foot, including anchorage reaction	1 980 000
Total on pier	4 378 700
Total on one pedestal 7 feet 3 inches square	2 189 350

$(7 \text{ feet } 3 \text{ inches})^2 = 7\,569$ square inches and $\frac{2\,189\,350}{7\,569} = 289$ pounds per square inch.

APPENDIX "C."

COLORADO RIVER CROSSING.

Line recommended *via* Red Rock Crossing.

3 miles new rails and fastenings (60-pound rail)	$16 060 00
9 500 new ties at 72 cents	6 840 00
Laying and surfacing 13 miles track	10 000 00
Grading 13 miles new line	89 315 00
Pile and trestle bridges	33 900 00
Masonry and iron work at Colorado River Crossing	241 210 00
Total via Red Rock Crossing	$397 325 00

Change of line, retaining present crossing, grading and protection of banks	$75 950 00
3½ miles new rails and fastenings	15 736 00
11 083 new ties at 72 cents	7 980 00
Pile and trestle bridging	10 037 00
Masonry and iron work for permanent structure, with draw span 160 feet clear	271 335 00
Estimated cost of raising grade at Powell	10 000 00
Riprap at ends of bridge	10 000 00
Total	$404 038 00
In addition to the above, there is a present charge of $6 000 00 per annum for demurrage, which is probably less than it would cost to rectify the channel and maintain a drawbridge. This amount capitalized at six per cent. per annum, would equal	100 000 00
Grand total via present crossing	$604 038 00
NOTE.—To use the present wooden bridge, it would be necessary to add 500 feet to its length and to construct a permanent iron draw span on the plan contemplated in the second item of this exhibit. This will require an outlay of $81 650 00, which, added to the cost of change of line, equals	214 353 00
If the dike is built before high water of this year, 500 feet must be added to the present bridge, at an expense of	17 500 00

Report signed by

A. A. ROBINSON,
Chief Engineer,
Atchison, Topeka and Santa Fé Railroad.

F. W. BOND,*
Resident Engineer,
St. Louis and San Francisco Railway.

Report made about 15th February, 1888.

* F. W. Bond acting for Chief Engineer James Dun on account of his not being able to attend.

APPENDIX "D."

Condensed Statement of Cost, Final, June 1st, 1890.

No.	Division of Accounts.	Caisson.	Masonry.	Preparatory.	Grading.	Pile Bridges.	Cross-Ties	Rails.	Track Fastenings.	Track Laying.	Superstructure.	Total.
A..	Labor	$71 754 02	$55 257 39	.....	$68 339 86	$17 274 04	.....	.....	$55 49	$12 584 43	$2 138 79	$227 404 02
B..	Timber	7 665 02	.....	.....	.....	13 069 93	.....	.....	.....	.....	1 684 68	22 419 63
C...	Piling	315 24	.....	.....	.....	10 717 32	.....	.....	.....	.....	.....	11 032 56
D..	Iron and steel	2 180 13	.....	.....	.....	2 550 91	.....	.....	.....	.....	213 537 83	218 268 87
E..	Tools and material	8 415 65	.....	.....	.....	.....	.....	.....	.....	.....	.....	10 210 57
F..	Fuel and water	7 158 30	.....	.....	.....	.....	.....	.....	465 49	703 85	625 58	21 964 80
G...	Cement	9 568 00	12 396 80	.....	.....	1 180 75	.....	.....	.....	632 66	1 340 94	16 800 00
H...	Cross-ties	.....	.....	.....	.....	.....	$16 800 00	.....	.....	.....	.....	30 100 00
I...	Rails	.....	.....	.....	.....	.....	.....	$30 400 00	.....	.....	.....	1 983 72
J...	Track spikes	.....	.....	.....	.....	.....	.....	.....	1 983 72	.....	.....	2 997 46
K...	Angle plates	.....	.....	.....	.....	.....	.....	.....	2 987 46	.....	.....	632 34
L..	Track bolts	.....	.....	.....	.....	.....	.....	.....	632 34	.....	.....	4 928 90
M..	Servis tie plates	.....	.....	.....	.....	.....	.....	.....	4 928 90	.....	.....	31 852 24
N..	Freight A. & P	7 641 17	8 250 00	$525 15	1 941 18	4 492 86	3 316 50	4 490 09	1 154 05	41 18	.....	29 514 04
O..	Freight G. S. & A. T. & S. F.	5 722 03	.....	.....	1 350 14	3 221 14	5 487 88	11 028 58	2 704 27	.....	.....	8 372 80
P..	Local and train service	2 790 96	1 235 52	.....	.....	1 927 85	.....	.....	.....	1 215 69	1 202 78	7 808 79
Q..	Soundings	.....	.....	7 808 79	.....	.....	.....	.....	.....	.....	.....	6 238 17
R..	Tramway and tracks	.....	.....	6 238 17	.....	.....	.....	.....	.....	.....	.....	900 00
S...	Demurrage	.....	.....	900 00	.....	.....	.....	.....	.....	.....	.....	7 313 58
T..	Track to quarry	.....	.....	7 313 58	.....	.....	.....	.....	.....	.....	.....	24 060 33
U..	Engineering	5 052 67	3 127 84	962 41	2 646 64	2 165 42	.....	.....	.....	481 21	9 624 14	
	Total	$128 263 19	$80 267 65	$23 748 10	$74 277 82	$56 600 22	$25 604 44	$45 618 67	$14 921 72	$15 659 02	$230 154 74	$695 115 47
A. & P. Construction account No.		4.	4.	4.	3.	4.	5.	6.	7.	8.	4.	Construction.

Total cost: Superstructure..... $230 154 74
Substructure..... 232 278 94

Total..... $462 433 68, being the sum cost columns 1, 2, 3 and 10.

The large amount ($23 748.10) for preparatory work was due to the natural difficulties of the situation above those ordinarily encountered in such work, including the rafting of material, building and maintaining tramway to caisson site, etc.

APPENDIX "E."

Specifications for Beton.

The beton, for filling the chamber of the caisson, crib above the chamber and the anchor piers, shall consist of English Portland cement, clean sand, gravel and broken stone, adding water (a minimum quantity), and mixed and proportioned in accordance with the directions of the engineer in charge. The broken stone shall be of rock of hard quality, broken to size of 2-inch cubes, and shall be washed clean of all dirt by throwing water over it before adding to it the cement and sand. Stone of one-half to 2 cubic feet in dimensions may be used in the caisson and piers in such manner as may be directed by the engineer in charge.

Mixing.—The cement and sand shall be mixed dry in convenient quantities and proper proportions by measure, and then wetted to the consistency of wetted meal, care being taken not to get too great a quantity of water; then add the broken stone, washed as mentioned before, the mass to be in place without delay, in courses not to exceed 8 inches in thickness, except where large stone are used in the structure, as in masonry, when two or more courses shall be used to fill in the course of masonry. The beton, before putting in place, shall be thoroughly mixed, either by hand with shovels or by machine adapted to that purpose, and immediately on being put in place shall be well and thoroughly rammed with iron rammers. If water is observed to rise and cover the surface of the course under the effect of the tamping, it will be taken as indicating too much water and the quantity of water will be reduced in the succeeding course. All courses shall be carried uniformly over the surface of the structure, taking care where "one man" stone are used to have no stone lay against a retaining plank, and that the space between be wide enough to secure perfect freedom in the use of a rammer.

Proportioning Cement in the Beton.—Referring to the proportion of cement to be used, there will be three qualities of work.

First.—For leveling of heavy pier on rock foundations where great strength is needed, the proportion will be one part English Portland cement to two parts of sand.

Second.—For filling the chamber of the caisson and for the first 6 feet of the crib above roof of chamber; for that portion of anchor piers

below and 2 feet above the anchor beams, and for the foundation of pedestals and shore abutment not exceeding 2 feet in thickness, one part of cement and three of sand.

Third.—For the upper portion of the anchor piers, for filling the crib of the main pier (caisson crib above first 6 feet), and for all foundation work over 2 feet in thickness, one part of cement to four parts sand.

Weight and Proportion.—Taking No. 1, and allowing a shrinkage in volume of all the different ingredients of 20 per cent., which is about what has been found by careful tests on half-yard samples, there will be required for each cubic yard in place 27 cubic feet + 20 per cent. (= 5.4 cubic feet) = 32.4 cubic feet of material per cubic yard. Fifty-five per cent. of this (17.8 cubic feet), or 5.9 barrels (nearly), will be broken stone. Forty-five per cent. should be cement and sand = 14.6 cubic feet, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement	4.86	= 1.62
Sand	9.74	= 3.25
Broken stone	17.80	= 5.93
Total	32.40	= 10.80

No. 1. Not to be used to exceed 6 inches in depth, except when used to fill between face stones and backing in masonry.

No. 2. On the same basis (*i. e.*, 32.4 cubic feet per cubic yard), we will have as before, 17.8 cubic feet of broken stone, and 14.6 cubic feet sand and cement, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement	3.65	= 1.22
Sand	10.95	= 3.65
Broken stone	17.80	= 5.93
Total	32.40	= 10.80

No. 3. Broken stone as before, 17.8 cubic feet, and cement and sand 14.6 cubic feet, divided as follows :

	Cubic feet.	Barrels of 3 cubic feet.
English Portland Cement	2.92	= 0.97
Sand	11.68	= 3.90
Broken stone	17.80	= 5.93
Total	32.40	= 10.80

It is considered practicable to use "one man" stone to the extent of from 25 to 40 per cent. of the total volume, thus saving a considerable quantity of cement. This refers, of course, to those portions of the work where mass and weight are required. When special strength is required, only the neat beton, as above, will be used. These "one man" stone, after being placed, should be wetted with a sprinkler to secure perfect adhesion to the beton when rammed between. The surface of all face stone and the backing in masonry should also be so wetted.

An approximate and uniform measurement of all material used is imperative. A half barrel measure is good, but the cubic foot measure is preferred.

The time in which the beton should be in place should not exceed forty-five minutes; less time is much safer. Water should be sparingly used and the mixing thoroughly done. A competent inspector will be placed on the work while it is being done, who shall see that all the directions are carried out and that the tamping is thoroughly done.

When each course has been thoroughly tamped and leveled off, it shall be immediately wetted thoroughly over its whole surface, to dissolve and smooth down all crumbs of mortar and dust so as to secure perfect adhesion of the succeeding course.

In the body of the main piers, beton will be used to fill between the face stone and the backing and between the stones of the latter, care being taken that the backing stones be so placed as to allow free use of the rammer between. Then, after the face seams are well filled with mortar and the surface of all stones wetted, the beton No. 1 will be deposited and tamped, using two or more courses as the depth of the course of masonry may require, until the same shall be fully leveled up to a true surface and bed for the succeeding course, and not omitting the thorough wetting of the surface, as required on the beton work before mentioned.

Clean gravel, sifted and assorted, with a minimum diameter of one-eighth inch to a maximum diameter of 2 inches, may be substituted for the broken stone in the concrete filling of the crib of the caisson. Only coarse, clean river or pit sand will be used. No wind drifted sand or such river sand as may contain earth, mud or dust will be allowed.

NOTE.—Observation in relation to the use of water during the progress of this work has tended strongly to confirm the correctness of the conclusions drawn by Gillmore and others as to the reduction of the amount of water used in this kind of work. The writer hereof, in some experiments in 1886, found that it was difficult in hand mixing to reduce the amount used below 14 per cent. of the mass, but in the work by the machine at Red Rock this was reduced to about 10 per cent., or about double that of natural absorption by solid rock. In this case an increase in amount of water used in mixing visibly reduced the strength of the work. It may be remarked, however, that the variation between different rocks or sands will have much to do with this question of the proper amount of water.—*Rowe.*

APPENDIX "F."

TESTS OF FULL SIZE EYE-BARS—STEEL.

SECTION.	Mark.	Ultimate Strain. Pounds per Square Inch.	Elastic Limit. Pounds per Square Inch.	Elongation.	Reduction of Area. Per cent.	REMARKS.
8 x 1 15/16"	*W*	41 490	31 660	4.1 per cent. in 23 feet.	.....	Crystalline, broke in head in flaw.
Same bar, reheaded and re-annealed	.....	56 360	34 600	10.5 per cent. in 20 feet.	49.6	Silky, broke in body.
8 x 2 1/8"	*A C B*	50 120	28 915	18.3 per cent. in 24 feet.	48.0	Silky, broke in body.
8 x 2"	*A C C*	48 345	27 570	15.8 per cent. in 24 feet.	53.7	Silky, broke in body.
7 x 1 3/8"	*C B*	52 460	32 100	12.3 per cent. in 25 feet.	57.1	Silky, broke in body.
8 x 1 1/2"	*C F C*	52 070	30 320	19.0 per cent. in 25 feet.	55.5	Silky, broke in body.
8 x 1 1/4"	*C D B*	57 050	32 870	16.4 per cent. in 25 feet.	32.6	Silky, broke in body.
7 x 2 1/16"	3 *A*	57 465	33 970	15.7 per cent. in 25 feet.	49.6	Silky, broke in body.
5 x 1 15/16"	*T O B*	52 220	32 530	11.4 per cent. in 20 feet.		Fifty per cent. granular. Broke in neck, in flaw.

DISCUSSION.

CHARLES MACDONALD, M. Am. Soc. C. E.—It is scarcely possible to discuss this paper from an engineering point of view, owing to circumstances which seem to have prevented a careful preliminary study of the problem. It is rather the record of a commercial enterprise than a professional report; and is an apt illustration of the old adage, "The more haste, the less speed." Had a complete set of borings been made, before any contracts were entered into, the subsequent proceedings would not have been complicated by obligations hurriedly undertaken, and which have resulted in an expensive and unsatisfying solution of a comparatively simple problem in construction. As to what might have been done in the way of designing a bridge for this location, which would have fulfilled all requirements at the least cost, it is not necessary now to take account.

The bridge described in the paper before us is doubtless designed with the proper margin of strength; and, in detail of connection, follows the best practice. There are, however, certain strategic principles of construction, involving general outline and subdivision of spans, which may be considered with advantage in this connection. One of these implies that lines of beauty should, of necessity, be lines of strength, while lines of strength may not always be lines of beauty. Nature, the wisest and best constructor, has established this law; and we may assume, as a corollary to it, that the best construction is that which combines scientific proportion with artistic excellence in design. If, therefore, the general outline presents abrupt and ungraceful lines of strength, it is evident that the law of correct proportion has not been complied with; and that an unnecessary expenditure of material and consequent expense has been involved in accomplishing the desired result. An inspection of the drawings would seem to indicate that harmonious outline was not a factor in the design to any considerable extent; and, for this reason, it is fair to assume that a minimum of cost has not been reached, although the safety of the structure is assured beyond question.

There are two questions of detail, which may be referred to in closing, viz.: Concrete in piers; and, steel inspection. In climates where frost does not exercise an injurious influence, it is believed that concrete might be utilized to a much greater extent than at present. The anchor piers of this bridge were of concrete; but the main piers were of masonry, presumably much the most expensive. The writer has built several river piers of concrete throughout, with good results.

As to surface defects in steel plates, for which cause large quantities of material were rejected, much has yet to be learned in this country. The writer has frequently had occasion to test plates which had been

rejected for so-called surface defects, and found them, in almost every instance, sound and good. The entire question of inspection should receive more careful attention, in order to insure harmonious working between the producer and consumer. In this connection, Mr. Alfred E. Hunt's paper on "Tests of Structural Steel," read before the American Institute of Mining Engineers, at the October meeting, 1891, contains many valuable suggestions.

W. H. Breithaupt, M. Am. Soc. C. E.—With the river bottom as it was supposed to be at the site of this bridge a cantilever design was without question the proper solution. But with the actual bottom as later found, a different design would have been more economical. For two 400-foot spans and a short span, all non-continuous, the cost of masonry would have been not much in excess of the actual cost. A pier in mid-channel would probably have cost less for the same height than the present east pier, considering the difficulties encountered and the changes that had to be made during its sinking; the two main piers, for through spans with rail elevations as used, would, however, have had to be higher. On the whole, as stated, the total cost of the masonry could not much have exceeded the actual cost. The weight of two 400-foot spans and a short span to make up the length out to out of the cantilever arms would be approximately 2 400 000 pounds. The actual weight as given is 3 400 000 pounds, a difference of 1 000 000 pounds. Erection would have cost a good deal more, and would also have been attended with more risk; but as the erection of a 400-foot span is a matter of a few days only when everything is ready, such risk would have been limited to the falsework. One million pounds at six cents per pound, the price mentioned in the paper, amounts to $60 000. Allowing, say, $20 000 for greater cost of erection and contingencies, reduces this to $40 000 as the difference in cost, had the survey on which the work was undertaken, established the actual conditions.

J. F. Wallace, M. Am. Soc. C. E.—One of the most interesting features of Mr. Rowe's paper, is the account of the difficulties experienced in determining the actual line of the bed rock at this locality. This is not the first time that engineers in charge of bridge surveys have been misled by boulder beds and have considered the bed rock to be at a higher elevation than it actually was; the true facts only developing upon the sinking of the caissons for foundations. A recent example of this kind was the Chicago, Milwaukee and St. Paul Railroad bridge over the Missouri River at Randolph, where the boulder bed was misjudged to be the solid rock. The original soundings at Sibley for the Atchison, Topeka and Santa Fé bridge over the Missouri River at that point, developed a boulder reef, which was at first supposed to be bed rock, but the soundings taken immediately before the construction of the bridge penetrated the various boulder strata and developed the true location of the bed rock before the substructure plans were made. It is not sur-

prising that the borings made at the Red Rock bridge in 1881, should have so misled the engineer, as the top of the boulder was at such a uniform level, that he was easily deceived in supposing the rock encountered to be the bed rock; but it is surprising that the second series of soundings taken in 1888, should have failed to disclose the fact that the east side of the bed of the river was composed of boulders and drift and should have failed to discover the true line of the bed rock.

In taking borings for an extensive and important structure, or, in fact, any structure that renders the taking of borings necessary to secure suitable foundations, the engineer in charge should carefully study the geological formation of the region in which it is to be erected, and the surroundings and conditions of the immediate locality. The borings should be carried to such a depth as to actually determine the character of the strata. In this case it does not seem that it would have been difficult to have determined the character of the rock encountered by the drill. The history of this bridge should certainly be a lesson, not only to engineers, but also to projectors and investors having control of enterprises of this kind. It is a common thing for capitalists to consider the money spent in engineering, particularly on the preparatory work, as an expense to be avoided, if possible, and in a large measure as money wasted; and this is a striking instance of where rush and hurry prevented proper examinations that would have saved a year's time and thousands of dollars to the corporation interested. One interesting item in Appendix "D" is the cost of the various soundings, which is noted as $7 808.79, a sum certainly large enough to have obtained accurate information as to the location and character of the bed rock, if it had been expended at the proper time and in the proper manner. The record from Mr. Rowe's paper would seem to indicate that the tools and appliances furnished for these borings were inadequate, and this fact seems to have been one of the causes that added to the expense of the borings and also to the unsatisfactory results obtained in this connection. The writer would like to mention a few facts in regard to the soundings taken for the Sibley bridge. Some of the soundings passed through several distinct boulders.

A large casing pipe was used with a steel shoe, and after drilling through the first boulder encountered, a smaller casing pipe was inserted, and by proceeding in this way, it was possible to drill through several boulders and through sand and gravel underneath. After the bed rock was reached, one boring was taken to the depth of 165 feet to determine accurately the strata at this point. Several other borings were taken far enough into the strata to determine accurately its character. This gave assurance that the intermediate borings would indicate the true elevation of the bed rock. The cuttings from the boring tools were carefully preserved and labeled and the location from which they were obtained accurately determined, so that the record shown was very com-

plete and was afterward verified in the sinking of several caissons. One interesting feature that was developed in the sinking of the caissons was the discovery of different boulders that had been encountered by the boring tools. Several boulders were discovered that had been entirely perforated by the drill and other smaller boulders were found that had been turned by the drill. One boulder was found about 12 inches in diameter that showed in two places where the drill had entered it for about 2 or 3 inches, and the drill had then turned the boulder partly in a gravel bed. The drill had then been removed and reset, when it struck the boulder nearer the center and passed entirely through it. Mr. Otto Sonne, M. Am. Soc. C. E., had the immediate charge of the party engaged in these borings, which were made under the direction of the writer as Resident Engineer, and O. Chanute, Consulting Engineer. The borings at Sibley covered the entire bridge site and were taken approximately at the four corners of the probable location of each pier, and also extended across the entire valley at intervals of about 1 mile.

The valley at this point was from 8 to 10 miles wide. The geological formations were, therefore, fully determined and no chances were taken. The writer does not remember the exact figures in regard to the cost of the borings at Sibley, as they were included in the general engineering account, but their total cost was approximately $5 000. Their accuracy contributed largely to the successful and economical prosecution of the work.

Edwin Thacher, M. Am. Soc. C. E.—The results of the investigation by S. W. Robinson, M. Am. Soc. C. E., to determine the most economic length for the center suspended span under different specifications, are interesting and instructive, as is also his investigation of the lateral stability of the cantilever as compared with a simple span of the same length ; but it is not clear to me, that because a simple span of a certain length and width has the same lateral deflection as a cantilever span of the same length and less width, the two have necessarily equal lateral stability. I have never considered lateral deflection a matter of any great consequence, as it will take care of itself; but have regarded as more important questions, the condition of the compressed chords when considered as trussed columns, and the resistance offered, to overturning by the wind. Considered as a trussed column the cantilever admits of a much less width for a given span than the simple truss ; but the reverse is often true as regards overturning, and I think that this latter consideration should govern in determining the width between trusses in cantilever bridges. As the author does not consider this question in his paper, the following calculation may be of interest. The surface exposed to the wind has been taken as twice the projected surface of one truss plus the floor surface, when the structure is unloaded, and at 9 square feet per lineal foot in addition, when loaded with empty cars

weighing 900 pounds per linear foot—1 square foot per linear foot of leeward truss being covered by the train. The weight of structure has been taken from the author's first Appendix B.

DEAD LOAD ON THE PIER:

	Pounds.
One anchor arm	484 935
One cantilever arm	459 670
Floor for anchor and cantilever arms	127 455
Suspension span, including anchor reaction	701 975
One-half excess of anchor arm over cantilever arm	12 632
Pedestal posts and bracing over piers	89 395
Track, including anchor reaction	297 000
Total dead load on one pier	2 173 062
" " " one pedestal	1 086 531

Moment of stability, bridge unloaded = 1 086 531 x 25 = 27 163 275 foot-pounds.

OVERTURNING MOMENT, WITH A WIND OF 30 POUNDS PER SQUARE FOOT:

	Foot-pounds.
One-half center span ..... 3 467 x 30 x 58.2 =	6 053 382
Reaction at anchor ..... 3 467 x 30 x 36.0 =	3 744 360
Anchor and cantilever arms ..... 8 352 x 30 x 44.9 =	11 250 144
Total overturning moment of the bridge unloaded ..... =	21 047 886

FORCE REQUIRED TO OVERTURN THE UNLOADED BRIDGE:

$$= \frac{27\,163\,275 \times 30}{21\,047\,886} = 38.7 \text{ pounds per square foot.}$$

When the bridge is loaded with empty cars, the moment of stability will be as follows:

	Foot-pounds.
From dead load (as above) ..... =	27 163 275
From train 450 x 660 x 25.0 ..... =	7 425 000
Total amount of stability, bridge loaded ..... =	34 588 275

And the overturning moment will be as follows:

	Foot-pounds
From the unloaded bridge (as above) ..... =	21 047 886
From a train on the center span = 270 x 165 x 48.3 =	2 151 765
From reaction at anchorage ..... = 270 x 165 x 36.0 =	1 603 800
From a train on both arms ..... = 270 x 330 x 48.3 =	4 303 530
Total overturning moment, bridge loaded .. =	29 106 981

FORCE REQUIRED TO OVERTURN THE LOADED BRIDGE:

$$= \frac{34\,588\,275 \times 30}{29\,106\,981} = 35.6 \text{ pounds per square foot.}$$

It is seen, therefore, that the wind specified is not sufficient to overturn the bridge. The anchor bolts also offer some resistance to overturning, so the width selected appears to be sufficient.

The effect of pin friction on the anchor bars appears to be important in a cantilever design. A curious effect of pin friction was observed in a bridge on the L. & N. R. R., built in 1867, and removed in 1891. The center pin in top chord rotated, making a complete revolution about once in three months, or about one hundred revolutions during the life of the bridge.

S. M. Rowe, M. Am. Soc. C. E.—Charles McDonald, M. Am. Soc. C. E., says: "It is rather the record of a commercial enterprise than a professional report." I presume he means the portion submitted by myself (as I can hardly conceive it applicable to the contribution by Professor Robinson and Mr. Quimby), and I assume such to be the case. Relative to his further criticisms, I agree with him in the main, but believe still that there may be exceptions—for instance, the "Forth Bridge"—and in relation to the use of concrete in the main piers, I would question its safety when, in addition to the weight of the structure, we have the wind stress on the whole structure, with an additional and dangerous element, "Torsion," acting on those piers.

Referring to the criticism of Mr. Breithaupt as to the cost of the two types of structure, I will only say that the result of our computations, made on the ground, differ from his; but to go through the whole matter would unduly extend this discussion, and may better be done in a separate paper.

J. F. Wallace, M. Am. Soc. C. E., says: "The engineer in charge should carefully study the geological formation of the region." To fully understand the situation at Red Rock bridge it is only necessary to repeat what I have already said. "The rock forming the bed rock of the river is volcanic." This, of course, settles any question of certainty, and the existence of any formation at one point gives no evidence of its extent or position.

The Colorado River is, perhaps, one of the worst rivers to deal with in the United States, and there are many points that may escape the attention of those unacquainted with it.

S. W. Robinson, M. Am. Soc. C. E.—The results of calculation by Edwin Thacher of the overturning moment of wind and the opposing moment of stability are certainly reassuring, and his conclusion of safe conditions is gratifying. In comparing his figures for the pressure on the main pier with those in "Appendix B (continued)," there appears to be some omission—perhaps the "expansion panel" and reaction, whereby the resulting moment of stability found by Mr. Thacher is about 10 per cent. too small. Further, taking account of the eight anchor bolts to each pedestal on the main pier, we gain approximately another 10 per cent. of stability. Adding these, we obtain the amount

of stability, 32 600 000, to counteract the wind moment of 21 050 000; or the equivalent of a wind pressure of 46 pounds per square foot. Again, for the loaded structure the 20 per cent. above should be added to the 34 588 275, making 40 000 000, to oppose the 29 107 000 wind movement; or the equivalent of 41 pounds per square foot of wind pressure.

As a further consideration to favor the stability of the bridge against overturning by wind, there can be no question but that the leeward truss will be to some extent shielded by the windward one; while 30 pounds per square foot will be conceded a fairly high figure for direct wind pressure. Just how much to deduct for the off truss may be questioned, but probably for this structure a fifth at least, whereby altogether the structure is found capable of resisting a wind pressure of 50 pounds per square foot.

Relative to pin friction, in the interesting case cited by Mr. Thacher, of a center pin in a top chord rotating one hundred times in twenty-four years, it is likely that the pin was not at all times under severe bearing pressure, from the fact that it was a center pin, and that rotation was due to the working of the parts across the slack in the holes, or possibly the jarring at a certain time during the passage of trains. Those familiar with the necessity of lubrication in machinery, of iron upon iron bearings, will hardly be willing to admit that a pin in a close fitting hole under severe pressure, as in anchor and suspension link pins, can be made to rotate one hundred times, dry, without stiffening to practical solidity by abrasion and cold welding. These conditions are quite different from those for the pin cited by Mr. Thacher. Suspension and anchor link and pin joints have been admitted into existing cantilever bridges under conditions which are believed to be positively dangerous, without lubrication, beyond a service of ten years. This is for short links, where a stiffened joint would strain parts beyond the elastic limit.

It is especially unfortunate that doubt should be thus thrown upon such vital parts as those here considered, because a failure could not do less than result in serious disaster. Hence these parts were deemed worthy of especial consideration and provision in the present structure. It is important that data on this subject from existing bridges should be collected and reported to the Society, giving examples, and particulars of deportment of the same in service.

H. H. Quimby, M. Am. Soc. C. E.—Mr. Macdonald's theory that artistic excellence necessarily accompanies the other virtues in a bridge is true, because such an object is judged by utilitarian principles, and what seems good is generally pleasing. The outlines of this bridge appear ungraceful because of the disproportion between the openings, a condition that was given, not designed. The members—which constitute the lines—are disposed so as to carry the stresses by the shortest roads to the piers, except where the trusses are deepened 10 feet over the main piers, the reason for which is given in the paper. Still greater

depth would have made it more imposing, but would have called for increased width. Curved chord lines would necessarily be made of a few straight panel chords, and would neither be beautiful nor compensate for the labor entailed. A better appearance to the miscellaneous eye might have been made by shortening the suspended span to 220 feet, making the lower boom of the cantilever arm similar in length and slope to the anchor arm, but it would have been at the expense of much additional metal in arms and anchorage.

Mr. Breithaupt presents an instructive comparison of what was, with what might have been done. His reference to the risk attending the use of falsework calls attention to a factor which must be recognized in estimating the cost of such work. As risk is a commodity with a more or less fixed market value, a proper percentage added to it will in some cases determine the adoption of a cantilever design, which experience teaches is safe and comparatively inexpensive to erect.

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